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THE UNIVERSITY OF ALBERTA

EFFECTS OF SALT CONTENT AND ADSORBED CATIONS ON THE SHEAR STRENGTH OF A REMOULDED HIGHLY PLASTIC CLAY SOIL

by

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A THESIS

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ABSTRACT

The specific problem that formed the basis of this thesis was the determination of the shear strength characteristics of a remoulded lacustrine clay that had been modified by partial saturation of the exchange complex with potassium, calcium, sodium and magnesium cations. The strength parameters were observed for two different salt concentrations of the pore water for each modification. The data were obtained from consolidated-undrained triaxial tests with pore pressure measurements augmented by triaxial consolidation, specific gravity and hydrometer analyses.

In order to provide a background, some material from the fields of clay mineralogy, physical chemistry and soil physics is included. A chapter dealing with a few case histories is also included to illustrate the practical application of the phenomena of cation exchange.

It is concluded that the shear strength characteristics of a clay soil are affected by changes in cations adsorbed or changes in the salt content of the pore water. The adsorbed sodium cation is responsible for a pronounced reduction of the angle of internal friction of the soil if the pore water is free of salts. The angle is reduced to a lesser extent for calcium but potassium and magnesium cations appear to have

little influence. An increase of the salt content of the pore water generally increases the angle of internal friction. This increase is most pronounced for the sodium modification. The presence of salts in the pore water also affects the time to theoretical one hundred percent consolidation for all modifications. It is generally concluded that the strength parameters of a cohesive soil are influenced by the salt content of the pore water and the nature of the adsorbed cations.

It is recommended that future research investigate more specifically the effects of salt concentration in the pore water and the effect of one cation on another as two species become adsorbed. Consolidation and swelling characteristics of the various modifications are also recommended research projects.

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CHAPTER I

INTRODUCTION AND STATEMENT OF PROBLEM

1.1 The origin of the idea for this thesis is to be found in a consideration of the landslide investigations in the pre-consolidated clay shales of northern Alberta, the foothills region of southern Yukon and northwestern British Columbia. Analyses of failed slopes in these areas, in terms of effective stresses with piezometrically measured pore water pressures in the field, yielded factors of safety much in excess of unity. An example of such a slope failure is given by Hardy (27)*. Another way of considering these slope failures is to work backwards from observed data and determine what excess pore water pressures (or piezometric heads) should have existed in order that a factor of safety of unity be found from effective stress stability analyses. Piezometric heads sufficiently high to show a factor of safety of one have not been observed in the field. In the search for some mechanism to explain this anomaly it was conceived that ion exchange offered one possible mechanism to account for the reduction in strength of these soils.

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* Figures in Parentheses refer to references in the Bibliography.

1.2 Consideration of the ion exchange idea led to the general problem on which this thesis is based and which may be stated as a preliminary investigation of inter-relationships between physical-chemical phenomena and the shear strength of a clay soil. There are many cations that may be investigated but the program that forms the basis of this thesis was restricted to the cations normally found in nature; namely; calcium, magnesium, sodium and potassium. In addition to cation exchange is the closely associated problem of a qualitative examination of the effects of the concentration of salts in the pore water. This latter aspect was not planned during the initial stages but arose as testing progressed.

1.3 The general problem of physical-chemical phenomena in shear strength studies is really twofold. The first is whether or not the adsorbed cation complex or salt concentration does influence the shear strength of a clay soil to a significant extent and the second is whether or not the findings of a laboratory program are applicable to field problems. This thesis deals with both phases of the problem, the first in some detail as it is felt necessary not only to be able to show qualitatively cationic effects but also to be able to suggest an explanation for the mechanism by which they affect the shear strength. If an explanation is to be given it must be capable of explaining effects of the

ion complex on other engineering (or geotechnical) properties of a soil. Therefore, some reference is made to properties other than strength. Based on the results and discussion of the first part of the problem brief consideration is given to the applicability of the results to field problems.

1.4 The explanations presented herein lean somewhat heavily on the fields of physical chemistry and soil physics. Thus, to be a coherent entity, some space must be devoted to a consideration of these subjects. Since cation exchange involves clay minerals these too must be briefly reviewed. The consideration of these subjects that may be considered alien to the civil engineer is justified first on the grounds that any explanation basic to the thesis that cannot be understood without reference to many other volumes or articles is apt to leave the reader more confused than enlightened. Second, is the viewpoint that sufficient information must be gleaned to place the entire subject in a proper perspective in which the interplay of forces can be viewed in 'three dimensions'. Third, and perhaps most important, is to present a partial literature review of the various disciplines involved from the point of Soil Mechanics in an endeavour to determine whether or not it is a fruitful field for meaningful research. The author, in preparing a background for this thesis has had rather close contact with the Departments of Chemistry,

Geology and Soil Science. There are many fields common to these sciences and Soil Mechanics; the major difference being that of emphasis placed on the results.

1.5 A preliminary investigation which doubled as pilot model tests, was carried out on natural undisturbed soil samples of a clay shale. The procedure used in this early work was based on the following reasoning. Many of the clay shales are in a state of rebound due to release of load resulting from the retreat of the glaciers and to land erosion. They are taking up water during the rebound and the water should contain cations in solution. The work was reported in the First, Second and Third Progress Reports on the Effects of Cations on the Shear Strength of an Alberta Clay Shale (see Appendix A). The results were promising in that strength differences were detected when the relative proportions of cations in the adsorbed complex were changed. However, the results may have been masked or influenced in an indeterminate way by purely natural variations in the undisturbed samples or by variations in the pore water salt content. One of the conclusions arising from this preliminary work was the necessity to work with remoulded, laboratory prepared samples in order that such variations as may occur can be attributed, with confidence, to cationic modifications. This preliminary work was also extremely helpful in determining workable procedures in succeeding

investigations. Many technical difficulties were eliminated and successful laboratory techniques were established.

1.6 Further, and contemporary, investigations by P. A. Thomson (62) in 1960 and Hamilton (26) in 1961 also contributed to the working out of laboratory techniques. Their conclusions revealed that the adsorbed cation complex has a considerable effect on the Atterberg Limits and consolidation characteristics of a clay shale. The literature searches carried out by both Hamilton and P. A. Thomson added to the background knowledge and placed this thesis in a much more hopeful light. Much is owed to the tireless efforts of Hamilton who, in cooperation with Dr. Pawluk of the Department of Soil Science, evolved a workable procedure for the production of an homionic soil.

1.7 Since this thesis is primarily concerned with strength studies due to cationic modifications there is some restriction placed on the scope of the work involved. It is inevitable, however, that other geotechnical properties must be briefly dwelt upon since there is a close connection between them and strength.

1.8 In summary, the general problem to be considered concerns the effects of physical-chemical phenomena on the shear strength of a clay soil. Specifically, the problem consists of determining the shear

strength characteristics of a remoulded lacustrine clay that has been modified by partial saturation of the exchange complex with sodium, potassium, calcium and magnesium cations. Triaxial shear testing will be augmented by hydrometer analyses and specific gravity determinations.

CHAPTER II

A BRIEF BACKGROUND OF CLAY MINERALOGY, PHYSICAL CHEMISTRY AND SOIL PHYSICS

2.1 In order that subsequent suggested explanations of the observed data may be more readily visualized, a brief review of the applicable concepts of the fields of Clay Mineralogy, Physical Chemistry and Soil Physics is desirable. . For those readers who are well versed in these disciplines, this brief review will suffice. For others, who feel that they would desire a broader background, more detail is provided in Appendices B, C, D and E entitled Clay Mineralogy, Physical Chemistry, Soil Physics and Origin of Clay Minerals respectively.

Clay Mineralogy

2.2 Most of the clay minerals have sheet or layered structures although a few have tubular or fibrous structures. Argillaceous material may be considered as being comprised of very small particles each of which is either a book of sheet-like units or a bundle of tubes or fibers. Individual clay soils may contain a mixture of book-like units or books and bundles along with other components such as the normal rock-forming minerals.

2.3 The fundamental building blocks of the clay minerals are the silica tetrahedron and the alumina octahedron. The silica tetrahedron consists of a central silica atom surrounded by four oxygens or hydroxyls in the form of a pyramid. The alumina octahedron may be considered as an aluminum atom surrounded by 6 hydroxyls in octahedral coordination. The silica or alumina atom may be substituted by other minerals and this phenomena is referred to as isomorphous substitution. The net negative charge of the clay minerals is satisfied by the adsorption of cations.

2.4 The main clay minerals are kaolinite, montmorillonite, and illite or hydrous mica. Kaolinite consists of a single layer of octahedrons with a superimposed layer of tetrahedrons intergrown in such a manner that the tips of the tetrahedrons and apices of the octahedrons form a common layer. The mineral kaolinite is a stacking of such units tied together by hydrogen bonding. There is very little substitution within the lattices hence the cation exchange capacity is in the order of 5 to 15 milli-equivalents per 100 grams of air dried soil (abbreviated m.e./100 gm soil).

2.5 The montmorillonite unit is composed of an octahedral sheet enclosed by two tetrahedral sheets such that the tips of the tetrahedrons

point inwards. In the mineral montmorillonite, the oxygen layers of the bases of the tetrahedrons are adjacent to each other which gives rise to a weak bond and excellent cleavage. An outstanding feature of this mineral is the ability of water molecules and cations to enter between the unit layers causing expansion of the lattice in the 'c' direction. There is always substitution within the lattice of montmorillonite resulting in a high base exchange capacity of 80 to 100 m.e./100 gm soil.

2.6 The structural unit of illite or hydrous mica is the same as for montmorillonite except that some of the silicas are always replaced by aluminums in the tetrahedral layers and the resulting charge deficiency is balanced by adsorbed potassium ions. These ions tie the structural units together very effectively. The mineral illite appears to be a gradational stage between kaolinite and montmorillonite which leads to the favouring of the term hydrous mica in the general classification.

2.7 Mixed layer minerals arise from the fact that most clays are composed of more than one clay mineral that may be mixed in any of several ways. There may be discrete interlayering of pure minerals and such particles may cleave easily, particularly through montmorillonite layers. Another mixture possibility is interstratification since layers of clay minerals are very similar. These mixtures are as stable as pure minerals.

Physical Chemistry

2.8 The concepts needed from the broad field of physical chemistry are relatively few and are more fully dealt with in Appendix C. An important idea is that of the various types of bonding that can occur in molecular units. Ionic bonding involves the outright transfer of electrons from one atom to another. The covalent bond is the sharing of electrons in pairs, each electron pair corresponding to a single valence. In certain compounds, a hydrogen already bound by two electrons to an atom may further coordinate two more electrons to form another bond. This is termed hydrogen bonding and is effective but is the weakest type of bond.

2.9 When a molecule is placed between two charged plates a phenomenon termed polarization occurs. The anode attracts the electrons and the cathode attracts the nuclei. If the polarization disappears when the electric field is removed, the dipole is referred to as induced and the molecule is termed non-polar. If the centres of positive and negative charge within the molecule do not coincide, then the molecule possesses a permanent dipole and is termed a polar molecule. Water is an example of polar fluid.

2.10 Intermolecular or Van der Waals forces consist of both attractive and repulsive forces. Attractive forces arise from attractions of

permanent and induced dipoles and the vibrations of electron clouds with respect to the nuclei of atoms in a molecule. A repulsive force arises from the approach of electron atmospheres and nuclei of the atoms of one molecule with those of another. Coulombic forces result from the electrical interaction of two charges. All of these forces are susceptible to mathematical expression.

Soil Physics

2.11 The behaviour of colloids is predominately a function of the surface area of the particles comprising the colloid. There are three factors influencing surface area, namely, particle size, particle shape and the material of which the particles are composed. Subdivision of a particle into many smaller particles increases surface area per unit volume; changing the particle from an equi-dimensional to disc shape further increases the surface area for a given volume. The classic illustration of the effect of the material comprising the colloid is the comparison of colloidal suspensions of gold and clay. The former are stable but suspensions of clay often must be chemically treated to ensure stability.

2.12 Grim (20) gives the causes of cation exchange as broken bonds at the edges of crystals, substitutions within the lattice structure and the possible replacement of the hydrogen of an exposed hydroxyl group

at the edges of the crystal by an exchangeable cation. For kaolinite broken bonds play the major part in exchange capacity. In illites broken bonds contribute strongly but there is also some isomorphous substitution. Broken bonds account for about 20% of the high exchange capacity of montmorillonite and the remainder is due to isomorphous substitution.

2.13 In addition to adsorbing cations, the clay particle also hydrates by the formation of hydrogen bonds between surface hydroxyls, or oxygens, and the water molecules. Succeeding layers of water are held by similar bonds to the preceding layer. In this manner an adsorbed water film, or hull, is built up around the clay particle. A thick adsorbed hull on the clay particle is associated with adsorbed sodium ions and a thin hull with calcium and magnesium ions. The presence of extraneous cations and anions (or salts) in the pore water has the effect of depressing the thickness of the adsorbed water film around a clay particle. The thickness of the water hull is dependent on the type of adsorbed cation and the charge density of the particle.

2.14 The adsorbed cations are not in contact with the particle surface but diffuse away from it in such a way that the concentration of cations decreases as the distance from the particle surface increases. This concept is referred to as the diffuse double layer. The inner of the two

layers is composed of the negative charge associated with the clay particle and some cations adsorbed so strongly as to be considered immobile. The outer layer is comprised of the mobile and exchangeable cations. The clay particle and its adsorbed cations is an electrically neutral system which may be considered to be bounded by the particle surface and the outer edge of the diffuse double layer. The distance between these boundaries is a direct measure of a quantity termed the zeta potential. The higher the zeta potential, the greater the distance between the two boundaries. For a more complete presentation of the zeta potential, the reader is referred to Appendix D.

2.15 The general applicability of the Law of Mass Action is assumed in the production of an homionic clay. The equilibrium that exists between the cations adsorbed by the clay particle and those in pore water is continually shifted toward the homionic state by successive replacement of the pore water by a solution containing only the ions that one desires to have adsorbed. One method for the measurement of the cation exchange capacity is a leaching procedure in which the exchangeable positions are filled with ammonia cations. A determination of the retained ammonia yields the exchange capacity.

Origin of Clay Minerals

2.16 Clay minerals are a secondary product of the weathering or

alteration of rock-forming minerals. They may be considered to form in-situ and may undergo transportation by the natural agencies of the geologic cycle. The B horizon of the agronomist is the zone of accumulation of materials removed from the upper horizon including a concentration of weathering products some of which are the clay minerals. Typical weathering processes include oxidation, hydration, hydrolysis and the formation of soluble salts. An example of the conversion of orthoclase to kaolinite is given in Appendix E. However, as well as being a product of soil weathering, sedimentation and metamorphism also play a part in clay mineralogy.

CHAPTER III

OUTLINE OF PAST PRACTICAL APPLICATIONS OF CATION EXCHANGE IN CIVIL ENGINEERING

3.1 The object of this chapter is to review briefly the relatively few jobs in Civil Engineering in which cation exchange has played a major role. The purpose is to indicate more strongly the influence of physical-chemical phenomena on the behaviour and geotechnical properties of fine-grained soils. The agronomist has been aware of the importance of cation exchange for over a century and has been intensively investigating it for at least 50 years. Engineers have only realized its value in the past 30 years and have only begun to appreciate its significance to any large extent in the past decade. The outlines of the cases involving cation exchange cited in the following paragraphs have the further purpose of providing a practical background and to suggest to the engineer that the rather alien material is of useful significance in understanding the behaviour of clay soils.

Treatment of a Gravel Road

3.2 One of the earliest direct field applications was carried out by

Winterkorn in 1933 on a gravel road in Missouri (70). The gravel surfacing material had a clay binder which, according to its Atterberg limits, should have been suitable. On the road, however, it behaved like a silt; softening in the rain and exhibiting a high permeability. Excessive amounts of water entered the subgrade which resulted in many surface failures. Laboratory tests indicated practically all of the exchange positions to be filled with calcium ions. Test sections of the road were treated with solutions of the chlorides of calcium, potassium and sodium. The latter two salts improved the road considerably but the calcium-treated section became even worse. Further laboratory tests indicated that the dispersive effects of the sodium and potassium cations were responsible for decreasing the permeability. The calcium caused further aggregation which made the condition worse. (It would appear that the replacement of adsorbed calcium by sodium caused an increase in thickness of the adsorbed water hull on the clay particles hence a decrease in the permeability).

Decreasing Permeability

3.3 As part of the San Francisco Exposition of 1940 a shallow lagoon was to form a scenic area. It was to be fresh water with evaporation and seepage losses replaced from the city water supply. Seepage proved to be beyond tolerable limits and a remedy was sought. A thicker

blanket of clay was not economical since the clay had to be hauled in. Lee (37) took advantage of cation exchange simply by replacing the fresh water by sodium-rich sea water. The latter was left in place until a seepage of about forty inches of the salt water had occurred. The salt water was drained and fresh water pumped in. Subsequent tests and observations showed the permeability of the treated blanket to be about ten percent of the original value. This decrease in permeability as sodium becomes adsorbed is also reported in the recent investigations of P. Thomson (62) and Hamilton (26).

Liming of Soils

3.4 In the field of Portland Cement stabilization of soils, it is advantageous to have calcium as the adsorbed cation rather than sodium. In the hardening process of cement, lime (C_aO) is released to re-enter the reactions at later stages. If sodium is present in the adsorbed phase, an exchange takes place in which calcium replaces the sodium. This removes calcium from the hydration process of the cement and results in much lower strengths. "Liming", that is the addition of a small percent of lime (1 to 3%) to the soil before addition of the Portland Cement, is often advantageous.

Hydrometer Suspensions

3.5 Another application of cation exchange that is more familiar to

the soils engineer is the stabilization of hydrometer suspensions by the addition of Calgon (sodium hexameta phosphate, $Na_6(PO_4)_3$). It is known (20) that calcium soils (i.e. those soils having a majority of the cation exchange positions filled with calcium) tend to form large particles and that these particles are attracted to one another to form flocs which settle out quickly. The reason for this is the low zeta potential of the calcium clays which allows the particles to come close together. The addition of the sodium salt (Calgon) has two effects. The first is a partial cation exchange with an accompanying increase in the zeta potential. The second effect is the removal of calcium cations by the formation of complex phosphates (64). The overall result is a stabilization of the suspension. The presence of quantities of salts in the pore water of some Alberta clays ($CaSO_4$ and $CaCO_3$) will act to offset the effects of the added Calgon and flocs may easily form unless intolerable amounts of Calgon are added. In these instances it may be necessary to wash these salts out before doing the hydrometer analysis. Another point of interest with regard to montmorillonite clays is that the adsorption of monovalent sodium ions may cause sufficient swelling to disrupt the clay particle. In any event, cleavage takes place with relative ease between the structural sheets of some clay minerals during the dispersion phase of preparing an hydrometer sample. Grim (16) concludes

that "Mechanical analyses of kaolinite and some illite clays give a fair picture of the composition of the natural clay. Similar analyses of montmorillonite and other illite clays measure little more than the degree of disaggregation." It would appear necessary that every sample be treated in an absolutely identical manner if comparisons between different soils are to be made.

Artificially Sedimented Clays

3.6 An extremely interesting set of experiments was reported by Bjerrum and Rosenqvist in 1956 (5). In these tests a series of artificially sedimented clays were produced with sedimentation taking place in salt water and in fresh water. The sensitivity* of the salt samples was about 5 when the salts were present in the pore water. Subsequently these samples were leached with fresh water and the salt content reduced from 32 to 5 grams per litre. The sensitivity increased from the original value of 5 to an average value of 110, with individual values ranging from 70 to 180. The liquid limit was reduced from 49% to 28%. The undrained shear strength of the leached samples was some 40% less than that of the unleached samples. The fresh-water clays were characterized by a dense structure and a relatively high shear strength with the

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* Sensitivity is defined as the ratio of the unconfined compressive strength in the natural state to that in the remoulded state at the same moisture content.

sensitivity varying from 5 to 6. In analysing this data Bjerrum and Rosenqvist attributed the decrease in strength to the pore pressure parameter (defined as the ratio of increase in pore pressure to increase in deviator stress under undrained loading). This, in turn, postulates high pore pressures which were considered plausible due to a partial collapse of the unstable soil structure. In reviewing this work, the author of this thesis suggests that consideration may be given to suppression of the zeta potential which is known to occur when there is a high salt content in the pore water. As the salt content is reduced, the zeta potential tends to return to a higher value with a consequent strength loss. These experiments serve to emphasize the part that physical chemical phenomena play in the behaviour of certain soils. This general problem of reduction in salt content leading to reduction in strength may also be a factor in the Leda clays of Eastern Canada (9).

3.7 Another interesting and rather important circumstance pointed out by Bjerrum and Rosenqvist arising out of the experiments discussed in the preceding paragraph is the fact that sedimentation in bodies of water may lead to different types of soil strata depending on the electrolyte content of the water. Sedimentation in salt water leads to flocculation due largely to the ion content of the water. The flocculation may also be aided by preferential adsorption of ions other than sodium (e.g.

calcium or magnesium) by the clay particles (33). As a consequence particles of all sizes tend to settle out at similar rates, that is clay flocs settle at rates roughly equal to silt particles. This leads to homogeneous deposits of high sensitivity. On the other hand, sedimentation in pure water would result in little, if any, flocculation. Particle size segregation would occur due to different settling velocities and one would expect the product of such sedimentation to be a gradual gradation of particle sizes from coarse at the bottom to fine at the top of the layer.

3.8 A suggested mechanism for the origin of varved clays arises from a consideration of the preceding ideas. During the summer months of the glacial epoch a mixture of silt and clay size particles was transported to the glacial lakes. The silt sizes settled out due to their size and shape but the clay particles must have remained in suspension. The reasons for the clay sizes not having settled may have been turbulence brought about by the inflowing water, density currents, wind, etc. and Brownian movement. Flocculation did not occur to a sufficient extent due to the turbulence and possibly too low a concentration of electrolyte. However, in the ensuing winter period relatively still water conditions occurred due to the ice cover. The salt concentration of the water may have increased due to evaporation during previous summer and due to the formation of ice in the following winter. Under the conditions of

quiesence and possible increased electrolyte content, settling of the clay sizes occurred. In this manner, the boundary between the coarse and fine layers of varved clays can be accounted for.

3.9 One should bear in mind that salt water does not necessarily imply a marine environment since there is a definite possibility that some of the lakes associated with the glacial ages may have had a sufficiently high salt content to bring about some degree of flocculation. The general subject of the formation of varved deposits is discussed more fully in geologic literature which is beyond the scope of this thesis.

Landslide in Japan

3.10 Matsuo (43) investigated a landslide near Kashio, Japan and concluded that cation exchange played a major role in the slope failure. An analysis of rain water entering the slope revealed a calcium ion content of 0.07 milliequivalents per litre while the ground water issuing from the toe of the slope contained 1.13 m.e. per litre. Shear strength tests were carried out which indicated the natural soil to be weaker than a calcium soil. Stability calculations based on the test results gave a factor of safety of the slope for the natural soil of 0.6 and the calcium soil of 1.3. Matsuo concluded that the initially stable slope was predominantly a calcium soil and subsequent leaching reduced the adsorbed

calcium which lead to strength reduction and ultimate slope failure. It was suggested that calcium salts injected at the top of the slope would maintain stability. There was no mention of the quantity of salts in the pore water or of actual determinations of the adsorbed cation complex thus no observations can be made other than the suggestion that the changes in zeta potential due to leaching out of the pore water salts may have been a factor in the slope stability.

Lambe's Equation

3.11 Lambe in a recent paper (35) presented the following equation as applicable to a saturated fine-grained soil system:

$$\sigma' = \sigma + A - U - R$$

where: σ' = effective normal stress on the shear plane

σ = total normal stress on the shear plane

U = stress in the pore water

R = repulsive forces between the clay particles

A = attractive forces between the clay particles.

The adsorbed cation complex will affect both the repulsive and attractive forces, probably by differing amounts, which will lead to strength variations. The higher the zeta potential the greater the repulsive forces hence the lower the strength. The converse would also appear to hold. If the zeta potential is suppressed by the presence of salts in the pore

water and then these salts are removed without a volume change taking place, the resulting repulsive forces must increase. If, however, volume changes do occur the repulsive forces change further and as a result so will the strength. As this topic will be referred to later this brief reference will suffice for the purposes of this chapter.

Laboratory Consolidation

3.12 It is felt that the work of P. A. Thomson (62) and Hamilton (26) on remoulded soils should be mentioned although they are not wholly practical applications. Hamilton worked with the same soil as that used for the work of this thesis and hence his results are directly useful to the author. As Hamilton's work is the more recent only his conclusions will be given. Briefly these were:

(a) The swelling index* for calcium and magnesium clays was sensibly the same as that of the natural clay, that for potassium was slightly higher and that of sodium clay was about 4.5 times as high as that of the natural soil.

(b) Both calcium and magnesium cations caused a similar small increase in the compressive indices. The sodium cation caused

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* The swelling index of a soil is defined as the tangent of the slope angle of the rebound branch of the log pressure-void ratio curve. It is a measure of the volume increase due to removal of pressure.

the compressive index to almost double and the potassium ion lowered the compressive index slightly from the natural soil.

(c) The effects of the cations on permeability was also pronounced in the sodium clay, it being about one-tenth as permeable as the natural soil. The potassium soil was about one-third as permeable and calcium and magnesium soils about the same as the natural soil.

Summary

3.13 One of the earliest field applications of cation exchange was that carried out by Winterkorn. The properties of the gravel surfacing material were improved by the addition of potassium and sodium salts. Lee successfully reduced the permeability of the lagoon lining in San Francisco Exposition by exchanging adsorbed cations for sodium. The stability of soil suspensions for hydrometer analyses is a physico-chemical phenomena involving cation exchange. The notable experiments by Bjerrum and Rosenqvist point out the significance of a change in salt content in the pore water in considerations of both strength and stress-strain characteristics. Their experiments also lead to considerations of the formation of soil strata from soil particles deposited in bodies of water. Matsuo concluded that cation exchanged played a major part in the failure of a slope in Japan. All of these cases as

well as the recent approach of Lambe serve to direct the attention of the soils engineer to the vital part played by physical chemistry in the behaviour and understanding of the shear strength of cohesive soils.

CHAPTER IV

SAMPLE PREPARATION AND SOIL TESTS

Soil Used in Program

4.1 The soil used for this testing program was obtained from a highway cut located on the southern outskirts of Edmonton. The cut was opened in the latter part of the summer of 1959 and the samples were dug out of the backslope just above road level in early April of 1960. Some 6 to 8 inches of surface material was cleared away and about 500 pounds of soil dug out. This spot was about 15 to 20 feet below the original ground surface. The material was dumped into a large shallow container in the laboratory and allowed to air dry. Geologically the material is a glacial lake sediment and it is classified as a highly plastic clay having a liquid limit of 75 and a plastic limit of 30 per cent. The soil in its natural state contains about 45% clay sizes, 53% silt sizes and the remainder sand. The probable mineralogical composition of clay size fraction as determined by x-ray diffraction carried out for the author by the Alberta Research Council is - Montmorillonite, 20 - 40%; Illite, 20 - 40%; Chlorite, 10 - 25%; Kaolinite, 10 - 25%. After air drying and mixing, random portions were broken down and crushed to

pass a No. 40 sieve. About 7000 grams were stored for subsequent use.

Preparation of Modifications

4.2 The preparation of homionic modifications was attempted to enhance any effects and to reduce the possible interaction of one cation on another. The following procedure is based on the work of Hamilton (26). Two hundred gram batches were weighed out and placed in one-quart size glass kitchen sealers. Eight hundred millilitres of a 0.75N hydrochloric acid was added slowly with constant stirring. The reaction with the carbonates was so marked that care had to be taken to avoid the solution bubbling out of the bottle. The violent effervescence subsided in about 10 to 15 minutes and the mixture was then placed on a milkshake mixer for one minute. The cover was placed on loosely and the mixtures allowed to stand for 24 hours. In this time the soil settled out and the supernatant solution was siphoned off. The jars were refilled with 0.75N HCl (500 cc) remixed on milkshake machine and allowed to stand. This procedure was repeated twice more. It was assumed that these four applications of acid were sufficient to remove any carbonates or sulfates. After siphoning off the supernatant HCl of the last acid wash, the jars were filled with one normal solution of the acetate of the desired cation. These were mixed on the milkshake

machine for a period of not less than one minute and then allowed to stand. Settlement was usually complete in 24 hours but on occasion the samples were allowed to stand for several days. On completion of settlement, the supernatant solution was siphoned off and replaced with fresh acetate solution. This procedure was repeated four times.

4.3 The grinding to pass a No. 40 sieve proved a tedious step and an attempt was made to eliminate this step. The soil was broken down to particles about one half inch in diameter then placed directly into the acid. They quickly broke down but, in general, this short cut was not considered successful due to the fact that the sulfate crystals (selenite) were not completely removed. When the selenite was ground to pass the No. 40 mesh sieve, it appeared that its solubility was enhanced sufficiently that it could be removed at least to the point where it did not seriously interfere with subsequent reactions.

4.4 After the last acetate washing, the supernatant solution was siphoned off and the remaining slurry remixed. Each jar was split among four 250 ml centrifuge bottles. Each centrifuge bottle was filled with ethyl alcohol and centrifuged on a four head centrifuge at 1800 rpm. Normally sedimentation was complete in 10 to 15 minutes. The clear solution was poured off and replaced with fresh alcohol.

The soil was brought back into suspension with a milkshake mixer. Centrifuging and replacement of the alcohol was repeated four times. The soil was removed from the bottles and dried at 55°C. This washing with alcohol was repeated for the soil in each quart sealer. A batch was considered to consist of the total soil from 3 such sealers. Each batch was crushed to pass a No. 40 sieve, thoroughly mixed and split into 3 portions. Each portion thus obtained was sufficient to make one triaxial specimen.

Moulding and Testing Triaxial Specimens

4.5 The soil for one triaxial specimen was combined with distilled water in sufficient quantity to yield a slurry with a moisture content about 5% in excess of the liquid limit. After thorough mixing, the slurry was placed in prepared 2 inch inside diameter lucite tubes. Preparation of the lucite tubes consisted of placing a 7 cm diameter Whatman No. 54 filter paper over a porous bronze filter disc and forcing the tube down over the disc. The filter paper acted to wedge the porous plate in place and also to keep it clean. Four strips of wetted filter paper were placed up the sides of the tube to aid in drainage. A loading cap with a filter paper and porous plate was placed on the top of the sample. This assemblage was set inside a three inch inside diameter lucite container and the static load arm was brought into contact with the top plate. The

dial gauge was adjusted and sufficient weights were placed on the pan to give about 0.3 tons per square foot on the specimen. The outside container was filled with water and consolidation allowed to take place. Dial reading were noted until it was apparent that primary compression was complete. The load was then increased to one ton per square foot and further consolidation observed. A plot of dial reading versus time was kept. This double consolidation step was necessary due to the fact that if the higher laod was placed directly on the sample it tended to squeeze out past the porous plates. The initial compression step prevented this happening.

4.6 When the plot of settlement versus the logarithm of time indicated secondary consolidation had been reached, the set-up was dismantled and the consolidated specimen pushed out of the tube. The filter paper was peeled off and the specimen was trimmed to 35.7 mm diameter by 80 mm long on the Geonor trimming accessories. Five wool wicks were inserted lengthwise in the specimen to within about 1 cm of the top. Diameter, length and weight measurements completed sample preparation. Consolidation and failure in the consolidated quick test were carried out on the Geonor Triaxial Soil Testing apparatus essentially as outlined in the Norwegian Geotechnical Institute Instruction (46) and by Andresen et al (2). The major difference was the backpressuring

of the sample prior to testing. Two kilograms per square cm.were added to the cell and through the pore pressure device to the specimen. The pore pressure system was rigged through a mercury manometer which allowed pore pressures to be read to the nearest 0.01 kg per sq. cm. Each specimen was back pressured 12 to 15 hours prior to testing. Immediately preceding the strength test a pore pressure reaction test was conducted. This consists simply of increasing the cell pressure by one kg per sq. cm and observing the pore pressure reaction. Most samples had a 90% reaction in two minutes. The rate of strain during testing was in the order of 2.5% of the original length per hour. Observations of stress, strain, and pore pressure were taken at sufficiently close increments of strain, to enable stress-strain plots to be carried as sample loading continued. Failure was taken at the maximum principal stress ratio.

4.7 After failure, the specimens were removed from the cells, weighed, measured and sketched. Final volumes were determined by mercury immersion. About the middle third of the sample was dried for moisture content values. The remainder of the specimen and the sample trimmings were air dried and were used for specific gravity determinations, hydrometer analyses and flame photometric and cation exchange determinations. The compression to failure of a triaxial

sample normally ranged over a period of six to eight hours excluding setting up, consolidation and dismantling. The setting up and dismantling, including weight and volume measurements, took about two hours each.

Specific Gravity, Hydrometer and Cation Analyses

4.8 The specific gravity and hydrometer analyses were carried out in accordance with ASTM procedures and the flame photometer and cation exchange determinations followed the procedure set forth by the Alberta Soil Survey, Canada Department of Agriculture. These latter procedures are outlined in detail by P. Thomson (62). Hydrometer analyses were allowed to run for three to five days. Cation exchange capacity determinations required about one hour of the late afternoon of one day and the entire following day. The laboratory facilities of the Alberta Soil Survey Section are such that twenty-four samples can be handled at one time. Samples were duplicated and often triplicated. Flame photometer analyses are most efficiently run with two people and a set of twenty-four samples requires about three-quarters of a day. The chemical reagents and equipment for these tests were kindly supplied by the Alberta Soil Survey Section. The various acetate salts for the homionic modifications were obtained by the Department of Civil Engineering.

CHAPTER V

PRESENTATION AND DISCUSSION OF TEST RESULTS

5.1 In this chapter the results of the tests are presented and tentative explanations of the actions are offered based on physical chemical concepts. Broader aspects will be reserved for the following chapter. The grouping of the data in this fashion is felt necessary in order that the results of the entire program can be viewed as an entity. The cation exchange capacity and flame photometer results are referred to as the discussion proceeds. The results of the analyses of the soil for this data are presented in Table 1.

Specific Gravity of Soil Solids, G_s

5.2 Table 2 lists the results of specific gravity tests determined by the standard pycnometer procedure. The variation in the results is significant and particularly when one considers that the natural soil has calcium and magnesium occupying 98% of the exchange positions. The two readily apparent factors influencing these values are the presence of salts in the pore water and the adsorbed cation complex. The following paragraphs consider these in turn as entirely separate effects.

TABLE 1

CATION EXCHANGE CAPACITY AND FLAME PHOTOMETER RESULTS

Modification	Exchange[1] Capacity	Group[2]	Percent Adsorbed of				Salts in Pore Water [3]
			Calcium	Sodium	Potassium	Magnesium	
Natural	28.2	-	84.5	0.5	0.8	14.2	73.6
Potassium	29.0	1	18.6	0.5	75.8	5.1	31.8
	29.6	2	1.7	0.0	95.3	3.0	5.1
Calcium	32.8	1	93.7	0.6	2.1	3.6	21.8
	32.8	2	95.6	0.7	2.5	1.2	9.8
Sodium	32.9	1	5.3	87.9	3.4	3.4	0
	32.9	2	12.6	80.7	3.6	3.2	3.8
Magnesium	32.6	1	9.0	0.6	3.1	87.3	1.8
	32.6	2	5.7	0.0	2.1	92.2	16.5

Notes:

1. Exchange capacity and salts in pore water given as milliequivalents per 100 gm air dried soil.

2. Group I consists of first 3 samples only for each modification of Table 5 (consolidated at 1.5, 2.75 and 4 kg/cm^2). All subsequent samples comprise Group II.

3. Salts in pore water calculated as difference between exchange capacity by distillation and total of flame photometer results.

Table 2

Specific Gravity of Soil Solids of Modifications

Modification	Specific Gravity		Salts in Pore Water, m.e/100gm of Soil.
	Individual Determinations*	Average	
Natural	2.77, 2.80, 2.80, 2.76	2.78	73.6
Potassium	2.70, 2.67, 2.74, 2.77, 2.72, 2.67	2.71	31.8
Calcium	2.73, 2.73	2.73	9.8
Sodium	2.80, 2.79	2.79	0
Magnesium	2.68, 2.68, 2.67	2.68	16.5

* Carried out on Group I Modifications only

5.3 The basic formula for calculating the specific gravity of soil solids is (61):

$$G_s = \frac{W_s G_t}{W_s + W_{b+w} - W_{b+w+s}}$$

where: W_s = weight of soil particles as measured

W_{b+w} = weight of pycnometer plus water

W_{b+w+s} = weight of pycnometer plus water plus soil as measured.

G_t = specific gravity of water at the temperature of the test.

For any given soil or soil modification soluble salts in the pore water will increase the weight of dry soil by the weight of the salts. Let the dry weight of soluble salts be dW. For small amounts of soluble salts there is practically no weight loss when they are dissolved in water. To clarify this statement further, if one gram of a soluble salt (e. g. NaCl) is added to 500 grams of water the total weight will be their sum, that is 501 grams. To get the specific gravity, the formula would appear as follows:

$$G_{sm} = \frac{(W_s + dW)_m \cdot G_t}{(W_s + dW)_m + W_{b+w} - (W_{b+w+s} + dW)_m}$$

$$= \frac{(W_s + dW)_m \cdot G_t}{W_s + W_{b+w} - W_{b+w+s}}$$

In which the subscript 'm' refers to a measured quantity. It is apparent that the presence of soluble salts in the pore water increases the observed value of the specific gravity of the soil solids. Compare, for example, the values for natural and calcium soils in Table 2. Since weights for use in this formula must be determined to 0.01 grams, the effect of soluble salts can be significant. Variations in amounts of soluble salts in the samples may account for some of the scatter of individual determinations recorded in Table 2.

5.4 If the salts are insoluble then they will act like another soil grain

and the specific gravity so determined will be an average value. Selenite (gypsum) has a specific gravity of 2.3 hence its presence in the soil would tend to lower determined value for G_s. Other compounds (e.g. iron) would have the opposite effect. Consideration of the above formula indicates the 'dw's' of the denominator do not cancel but that both numerator and denominator increase. There will be a change in G_s but it will likely be small.

5.5 If one considers the weight of a sample of oven dry soil then a definite volume should be associated with this weight. If this sample is placed in a pycnometer, the resulting volume will be smaller than anticipated due to the dissolving of any soluble salts present. The salts going into solution will result in an observed specific gravity that is too high due to the denominator of the specific gravity equation being too small. On the other hand, due to hydration of the cations and the clay particles the volume will be greater than anticipated. As a consequence of the increase in volume of the sample the observed specific gravity will be too low. Undoubtedly, both of these phenomena act at the same time; one tending to counteract the other.

5.6 Another factor that may influence the specific gravity of the soil particles is the weight of the adsorbed cations. By multiplying the

cation exchange capacity of the soil by the weight of one milli-equivalent of the adsorbed cation, the total weight of the amount of a given cation adsorbed may be determined. The results of such calculations indicate that 100 grams of the soil used in this program will adsorb 1.23 grams of potassium or 0.63 grams of calcium or 0.72 grams of sodium or 0.38 grams of magnesium. These figures suggest that the potassium soil should be the heaviest and the magnesium soil the lightest for the condition of no salts in the pore water.

5.7 There is an interplay of all the factors discussed to such an extent that it is difficult to evaluate the effects of each factor separately. Further, there is insufficient data to assess which of the factors is predominant for a given set of conditions. The hydration of the soil particles and the adsorbed cations as well as the amount of the cations adsorbed are factors that depend to a large extent on the surface activity of the soil. Therefore, it is the clay soils, particularly montmorillonites, that would reveal the largest variations in observed specific gravity.

5.8 Changes of the adsorbed cations or concentration of salts in the pore water will lead to variations in the value of the specific gravity and, indirectly, calculations of the void ratio and degree of saturation. Thus,

if water is percolating through the soil along preferred paths, for example tension cracks or thin silt seams, the specific gravity is changing due to salt removal hence calculated void ratios are apt to be in error unless determined on soil samples having a salt content corresponding to that *in-situ*. In reality this is another way of saying that the void ratio is changing due to a change in the adsorbed water film. The point is that the specific gravity is used to calculate the value of the void ratio. Another interesting factor pointed out by Grim (22) is that the exchange of one cation for another may have no effect on the adsorbed layer up to a certain ratio of adsorbed cations at which point there is an abrupt change in the thickness of the adsorbed layer.

Hydrometer Analyses

5.9 To prevent flocculation of the soil in suspension during the hydrometer analysis a chemical is added. In the Alberta area the chemical is usually Calgon (sodium hexametaphosphate, $Na_6(PO_3)_6$) which has the effect of increasing the zeta potential presumably due to adsorption of the sodium cation. The very act of adding any chemical adds a large number of ions to the suspension in addition to any present in the soil sample. Since the suspension can be made stable, it follows that the salts in the pore water of the original sample have little influence on the hydrometer analysis. This statement must be taken as applicable

to 'normal' samples. There are soils with high salt contents whose suspensions cannot be stabilized unless intolerable amounts of Calgon are added. (See Paragraph 3.5). None the less, it appears reasonable to assume that, in a concentration of about 50 grams of soil per litre of suspension, the zeta potential can develop freely to its maximum extent. If this assumption is not strictly valid then at least there is much less suppression of the zeta potential in a five per cent suspension than there is in the soil *in-situ.* Manipulations of the formulas used in the hydrometer analysis with various assumptions concerning variation of the water film thickness and specific gravity for a given size of solid particle indicate that the influence of the adsorbed cation complex on the water hull of a clay particle does not have a very significant influence on the resulting grain size curve. However, the results of the tests shown on Plate 1, reveal a considerable difference in the particle size distribution for the various soil modifications. Grim (24) concludes that the reason for the increase in the percent clay sizes is the greater tendency for particles to break down along cleavage planes during dispersion when sodium occupies a high per cent of the exchange positions. (See also Paragraph 3.5 and Appendix B, Paragraph B.12).

5.10 It is pertinent to ask what prevents flocculation in the average hydrometer analysis if the zeta potential is not radically altered as it is

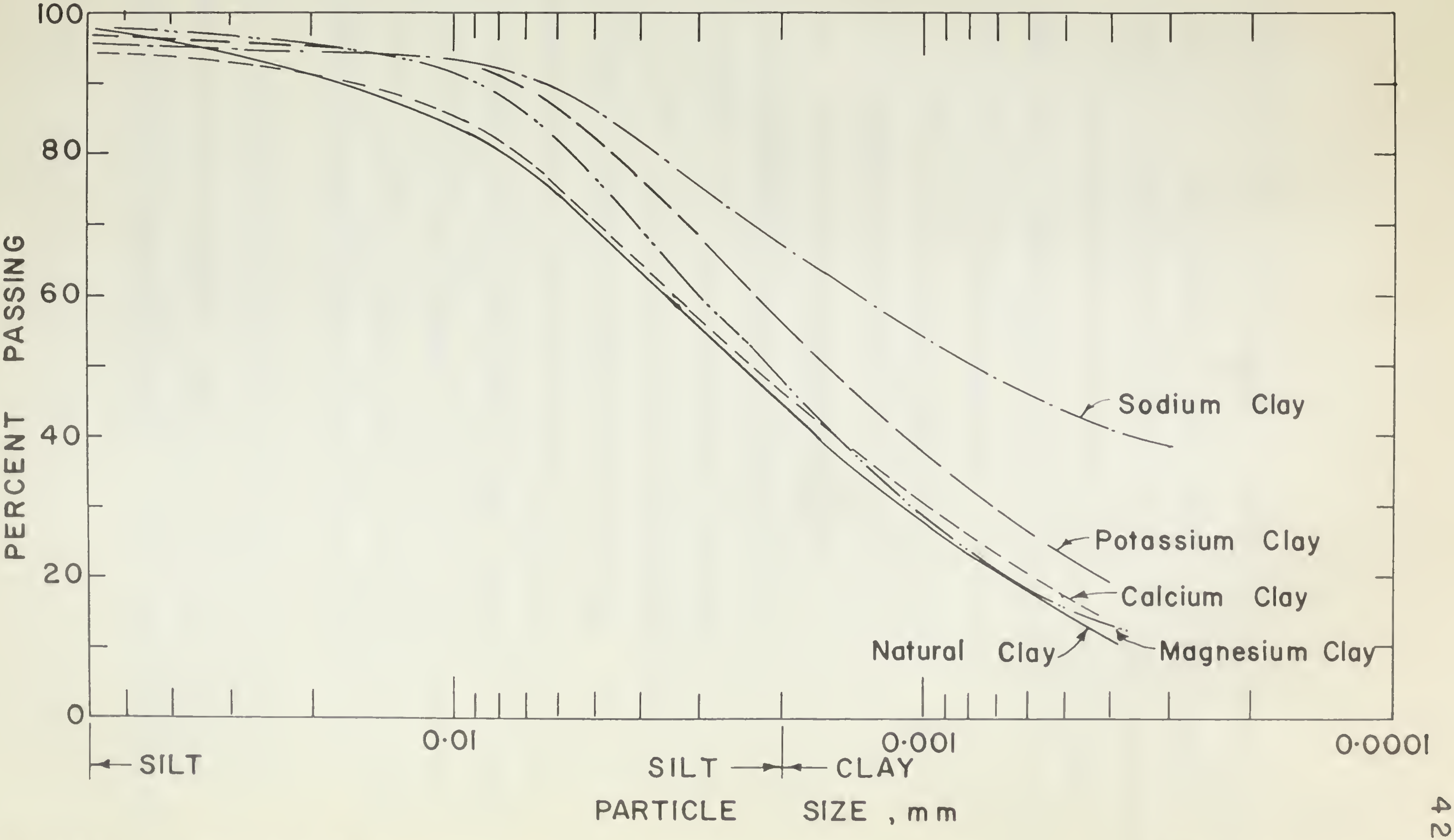

PLATE I PARTICLE SIZE DISTRIBUTION FOR THE CLAY MODIFICATIONS

in homionic clays. Ten milliliters of a 6% Calgon solution in a liter of suspension is approximately a 0.0009 N solution. If the results of Figure D4, Appendix D are applied then exchange of cations takes place to the extent of about 6% of the adsorbed positions being filled by the sodium cation. (There would actually be less than this since the results of Figure D4 are only directly applicable to a 10:1 water-soil ratio and not to a 20:1 ratio as is common in hydrometer analyses). This would tend to increase the zeta potential to some extent but there are other factors. Van Olphen (64) investigated the stability of montmorillonite sols and concluded that there are at least two different causes for the sol stability. First there is adsorption of the phosphate anion to the positive ends of the platy micelles (see Paragraphs D29 and D30 of Appendix D) resulting in a negative charge at the edges. This anion largely prevents the type of edge to surface association which leads to the formation of flocs in the presence of salts. Secondly, for systems containing dissolved calcium part of the stabilizing effect may be attributed to the inactivation of the strongly flocculating calcium ion by the formation of complex soluble compounds with the phosphate anion.

5.11 From the work of Grim (20) it would appear that as the zeta potential increases the distance between the unit layers of the clay mineral montmorillonite also increases. Larger spacing between unit

layers results in cleavage taking place more easily. It follows, therefore, that for a soil containing some montmorillonite, the per cent clay sizes as determined by a hydrometer analysis will vary directly as the zeta potential of the adsorbed cation. The data of Table 3 tends to confirm this postulate.

Triaxial Consolidation

5.12 During triaxial consolidation a plot of burette reading (burette connected to the sample) versus the logarithm of time was kept for each sample. Table 4 lists the times to theoretical 100% consolidation obtained from these plots. Typical plots are illustrated in Plate 2, plots for each test are contained in the applicable appendix (Appendices F to J inclusive).

5.13 The time-rate of consolidation is strongly influenced by the cations adsorbed on the clay complex as may be seen by a general comparison of the data in Table 4. For a given soil modification, the salt content of the pore water is also a major factor in determining the time for consolidation to take place. In order to facilitate comparison of the data at equal salt contents, the average consolidation time versus the salt content has been plotted on Plate 3 and the curves extrapolated to zero salt content of the pore water. This yields the following order

Table 3

Zeta Potential* and Percent Clay Sizes from Hydrometer Analyses

Modification	Natural	Ca	Mg	K	Na
Percent Clay Sizes	45	46	48	57	68
Zeta Potential, mv	—	52.7	54	56.5	57.7

* Zeta Potentials obtained from Baver (3)

Table 4

Time in Minutes for 100% Theoretical Consolidation in Triaxial Cell

Cell	Modification								Natural	
	Potassium		Calcium		Magnesium		Sodium			
Press	Time	S.C.*	Time	SC	Time	SC	Time	SC	Time	SC
1.5	9,700	31.8	500	21.8	390	1.8	39,000	0	180	73.6
2.75	14,000	31.8	150	21.8	300	1.8	40,000	0	130	73.6
4.00	13,000	31.8	170	21.8	170	1.8	46,000	0	130	73.6
5.25	3,900	5.1	140	9.8	150	16.5	39,000	3.8	160	73.6
6.5	3,900	5.1	90	9.8	170	16.5	40,000	3.8	140	73.6
7.5	4,500	5.1	150	9.8	150	16.5	22,000	3.8	140	73.6

*S.C. - Salt content in the pore water, m.e. per 100 gm air dried soil.

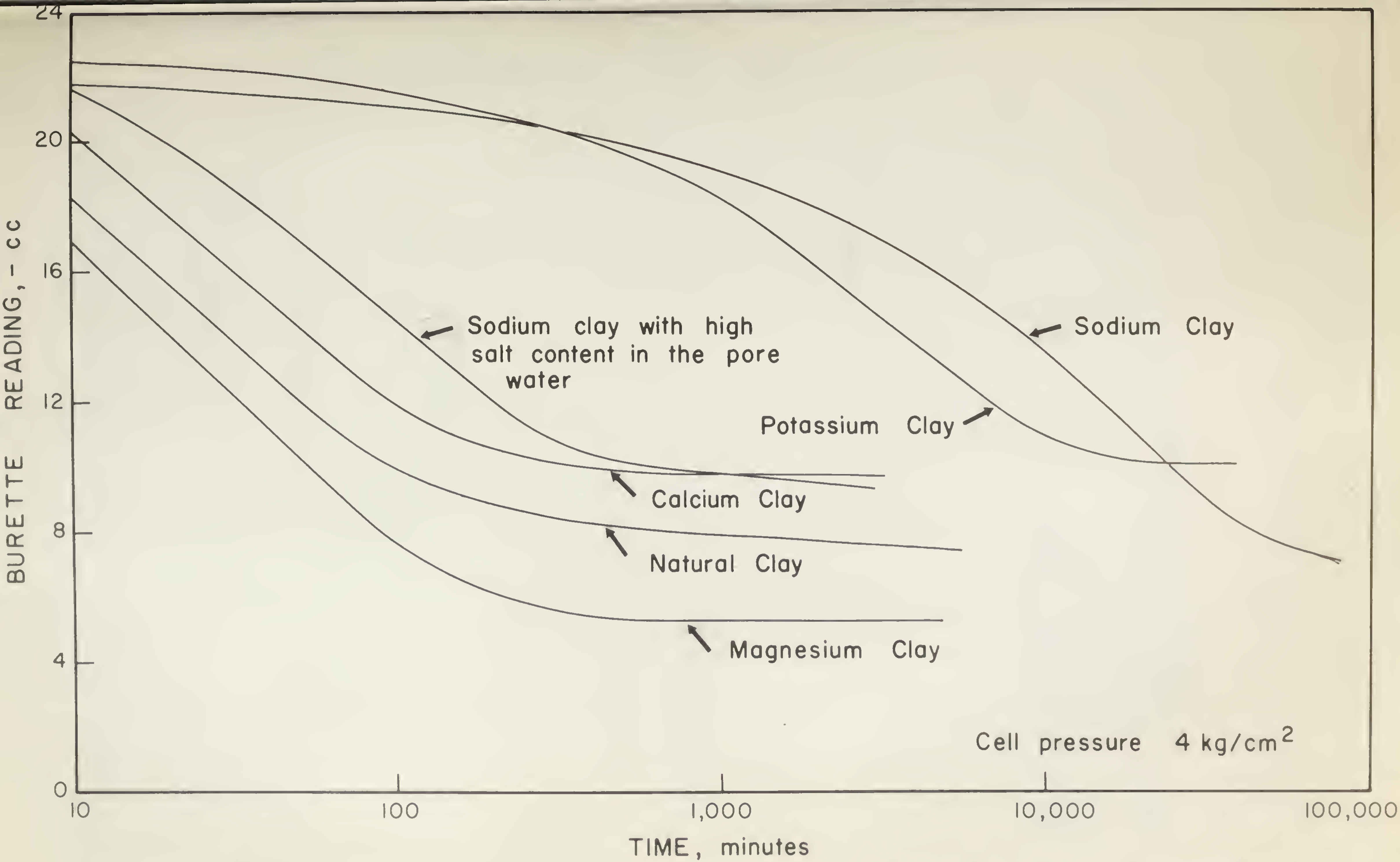

PLATE 2 TRIAXIAL CONSOLIDATION CURVES FOR THE CLAY MODIFICATIONS

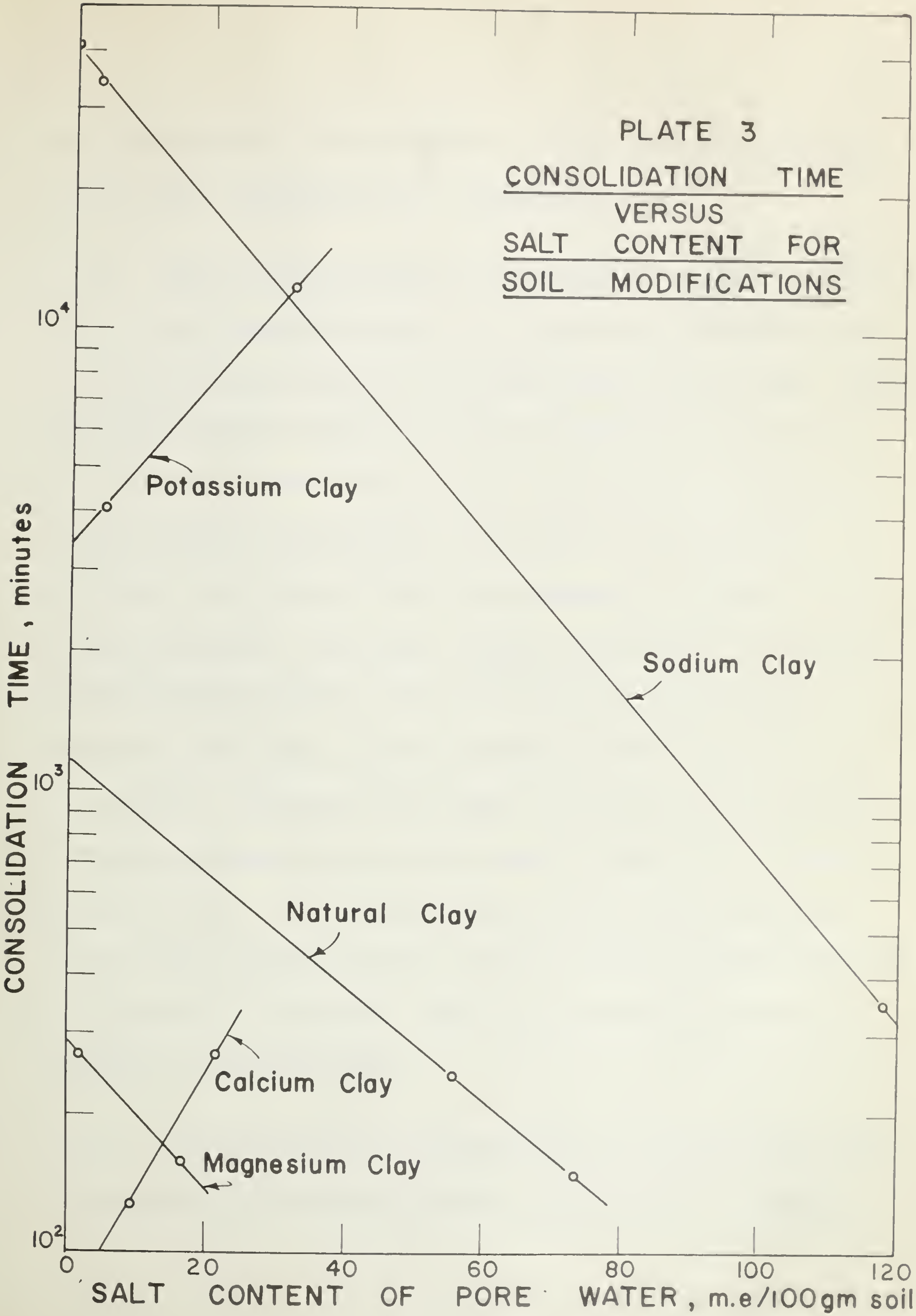

PLATE 3

CONSOLIDATION TIME VERSUS SALT CONTENT FOR SOIL MODIFICATIONS

for decreasing time for consolidation: Sodium clay - potassium clay - natural clay - magnesium clay - calcium clay.

5.14 The consolidation times for the condition of no salts in the pore water may be explained largely by a consideration of Poiseuille's Law which governs the velocity of flow through small diameter tubes. Taylor (61) shows that the diameter of a 'tube' through which water flows in a soil mass is related to the square of a representative grain size and that as the grain size decreases, the velocity of flow decreases. A decrease in the velocity of flow is accompanied by an increase in the consolidation time. From this concept it follows that the consolidation time should vary directly as the percent clay sizes in the sample. For the soil modifications, the data presented in Table 5 indicate that this is the case. In the case of the sodium soil there is another factor of importance, namely, the very thick adsorbed water hulls associated with this cation. It is suggested that these water hulls overlap one another hence reducing the size of larger pores and perhaps blocking off the smaller ones completely. Both of these phenomena increase the observed consolidation times.

5.15 For soils containing montmorillonite, the clay sizes vary as the zeta potential of the adsorbed cations. (See Table 3). This latter

TABLE 5

COMPARISON OF PERCENT CLAY SIZES, CONSOLIDATION TIME AND ZETA POTENTIAL

Modification	Time to 100% Consolidation Minutes	Clay Sizes per cent	Zeta Potential millivolts
Sodium	42,000	68	57.7
Potassium	3,400	57	56.5
Natural	2,000	45	----
Magnesium	280	48	54
Calcium	75	46	52.7

value is included in Table 5 and, for the modified soils, also shows a good correlation in that an increase in the zeta potential results in an increase of time to theoretical 100% consolidation. Plate 4 is a plot of zeta potential versus the consolidation time at zero salt content of the pore water (obtained from Plate 3).

5.16 The natural soil presents an anomaly in that it has the lowest percent clay sizes and a rather high consolidation time. The precise reasons for this are not known but the following facts may have a bearing on the matter. Both magnesium and calcium cations in relatively high amounts (84% Ca, 14% Mg) are present on the clay complex. The inter-effects of these two cations may lead to somewhat thicker adsorbed water

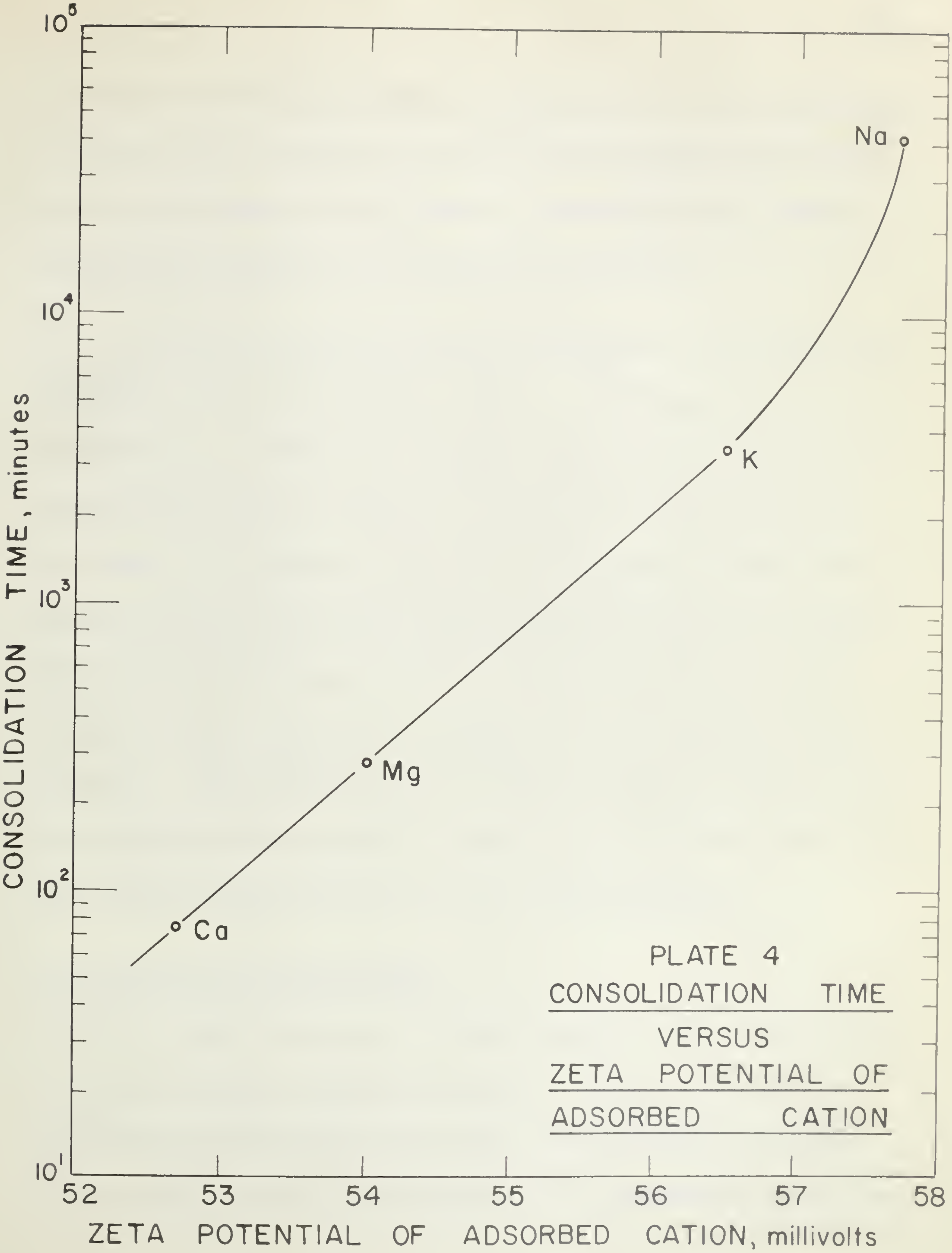

PLATE 4
CONSOLIDATION TIME
VERSUS
ZETA POTENTIAL OF
ADSORBED CATION

hulls than would be the case for an homionic state. As the salt content of the pore water increases, the adsorbed water films become thinner and the consolidation time decreases. It is possible that a similar reasoning also applies to the magnesium modification.

5.17 It is rather difficult to explain the behaviour of the calcium and potassium soils as is revealed on Plate 3. In the light of the previous explanations one would have to postulate either a thickening of the adsorbed water films or a decrease in particle size as the salt content increases. A decrease in particle size would appear unlikely due to the nature of the cations, thus the former postulate must be accepted. The increase in thickness of the adsorbed water hull must be small since the strength results are not suggestive of increasing water hull thickness. The structure of the soil mass is thought to become more flocculated as the salt content increases and this too should lead to decreasing consolidation times as the salt content increases.

Consolidated-Undrained Strength

5.18 The data observed for each test consisted of the deviator stress and the pore pressure in the water phase. From this the major and minor principal stresses and their ratio were calculated. Running plots of these five items versus percent axial compressive strain were

carried as the test progressed. A summary of the data from the triaxial tests is contained in Table 6. Data sheets and plots for individual test results are in the appropriate appendix (Appendices F to J inclusive). Typical plots for a cell pressure of 4 kg per sq. cm. are on Plates 5 to 9 inclusive. A study of these plates in order shows the following general points:

(a) Plate 5 - Deviator Stress versus Strain. The sodium modification is much weaker than the other modifications and the peak stress occurs at a much lower strain. This effect is just as pronounced at other cell pressures. The other modifications are reasonably well grouped.

(b) Plate 6 - Pore Pressure versus Strain. The pore pressure developed for the sodium sample is much lower than the others which again are reasonably grouped. Postulates to explain this phenomena are discussed in ensuing paragraphs.

(c) Plate 7 - Major Principal Effective Stress versus Strain. Sodium results are now within the group and not lower as might be expected from the low deviator stress. It appears that the low pore water pressures are responsible.

(d) Plate 8 - Minor Principal Effective Stress versus Strain. As a consequence of the pore pressures developed, the minor principal

effective stress is higher in sodium than in the other samples.

(e) Plate 9 - Principal Stress Ratio versus Strain. As a consequence of a high minor principal effective stress, the stress ratio is lower for sodium than for the other modifications and the maximum occurs at a much lower value of strain.

Tentative Explanations for Strength Observations

5.19 The questions to be considered arising from the preceding observations concern the low deviator stress and pore pressure for the sodium modification. A tentative explanation for these phenomena starts with the concept that thick water hulls are associated with the sodium modified soils (22). These water hulls must be in contact or somewhat intergrown one with another as is shown by the consolidation results. Since the oriented water layers extend relatively far from the clay particle surface, the orientation of the water hull at the outer edges is probably not very strong. The monovalent sodium ion also is not capable of tying adjacent particles together. The thickness of the adsorbed water hull increases the distance between particles hence reducing interparticle attractions and increasing repulsions. These three concepts lead to the conclusion that the shear strength of the sodium modification must be low since the water net may easily be disrupted at its outer fringes, there are no interparticle bonding forces due to the

TABLE 6

TRIAXIAL TEST RESULTS

Modification	Deviator[1] Stress	Triax. Cell Press.	Minor Prin. Effect[4] Stress	Major Prin. Effect Stress	Pore Press.[4]	Strain[2] at Failure	Water Content	Void Ratio	Degree of Saturation	Ratio Pore Press. to Dev. Stress
Natural	1.20	1.5	0.73	1.93	.77	8.75	42.3	1.20[3]	-	64.1
	2.01	2.75	1.01	3.02	1.74	8.60	38.3	1.09	97.5	86.8
	3.32	4.00	2.43	5.75	1.57	9.75	34.4	0.96	99.2	47.2
	3.35	5.25	2.27	5.62	2.98	10.0	35.3	1.01	97.0	89.0
	4.09	6.50	3.04	7.13	3.46	10.0	33.3	0.95	97.1	84.6
	3.92	7.50	3.42	7.34	4.08	10.5	32.7	0.93	97.2	104.1
	2.81	4.00	1.60	4.41	2.40	13.5	36.6	1.04	98.1	85.4
Potassium	1.10	1.50	0.73	1.83	0.77	7.25	45.6	1.25[3]	-	70.0
	1.90	2.75	1.29	3.19	1.46	9.80	40.9	1.13	99.1	76.8
	2.40	4.00	1.77	4.17	2.23	10.50	39.2	1.08	99.3	92.9
	3.45	5.25	2.12	5.57	3.13	12.0	35.1	1.00	96.1	90.7
	4.53	6.50	2.73	7.26	3.77	12.0	33.5	0.93	98.6	83.2
	4.72	7.50	3.18	7.90	4.32	12.0	32.8	0.91	99.1	91.5
Calcium	0.96	1.5	0.64	1.60	0.86	6.50	46.2	1.26[3]	-	89.6
	1.79	2.75	1.10	2.89	1.65	8.50	40.9	1.13	99.2	92.6
	2.47	4.00	1.97	4.44	2.03	7.50	37.6	1.03	99.3	81.5
	3.33	5.25	2.18	5.51	3.07	11.0	38.5	1.07	98.1	92.2
	3.65	6.50	2.84	6.49	3.66	9.0	36.3	1.01	98.6	100.4
	2.58	4.00	1.55	4.13	2.45	11.0	39.7	1.11	98.0	95.0
	4.98	7.50	3.66	8.64	3.84	8.5	30.1	0.86	98.3	77.1

TABLE 6 (continued)

Modification	Deviator[1] Stress	Triax.[4] Cell Press.	Minor Prin. Effect[4] Stress	Major Prin. Effect Stress	Pore Press.[4]	Strain[2] at Failure	Water Content	Void Ratio	Degree of Saturation	Ratio Pore Press. to Dev. Stress
Sodium	0.74	1.50	1.06	1.80	0.44	3.20	69.7	1.94[3]	-	59.4
	1.00	2.75	2.06	3.06	0.69	1.50	59.1	1.70	96.7	69.0
	1.37	4.00	3.16	4.53	0.84	3.20	54.7	1.56	97.8	61.3
	2.11	5.00	3.52	5.63	1.48	6.50	46.2	1.32	97.4	70.1
	2.73	6.50	4.51	7.24	1.99	4.75	42.8	1.23	97.0	73.0
	3.13	7.50	5.10	8.23	2.40	7.20	39.5	1.15	96.0	76.7
Magnesium	0.93	1.50	0.57	1.50	0.93	7.0	46.2	1.24[3]	-	100.0
	1.63	2.75	1.04	2.67	1.71	11.20	39.9	1.07	99.9	104.9
	2.49	4.00	1.71	4.20	2.29	11.25	35.1	0.96	98.5	92.0
	3.08	5.25	2.33	5.41	2.92	12.0	33.8	0.93	97.4	94.8
	3.75	6.50	2.86	6.61	3.64	13.50	34.0	0.92	98.8	97.1
	4.53	7.50	3.38	7.91	4.12	12.50	32.5	0.86	101.0	90.9
	1.75	2.75	1.19	2.94	1.56	8.50	39.5	1.04	101.0	89.5

Notes

1. All stresses in kg/sq. cm.
2. Strain, degree of saturation and the ratio pore pressure to deviator stress are in percent.
3. This figure is based on void ratio equals (water content) times G_s.
4. Abbreviations

Prin - Principal	Press - Pressure
Effect - Effective	Triax - Triaxial

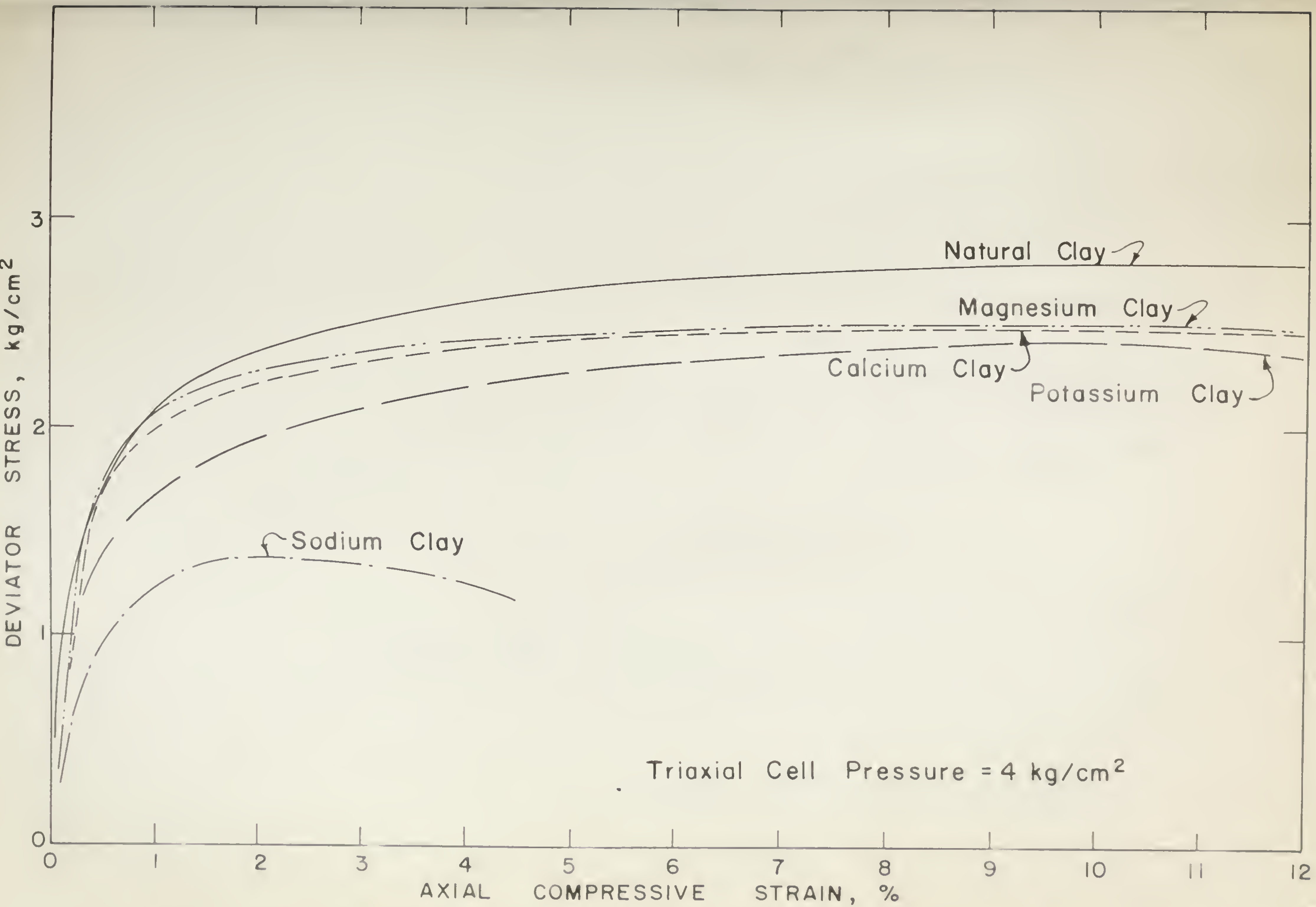

PLATE 5 STRESS-STRAIN RELATIONSHIPS FOR THE CLAY MODIFICATIONS

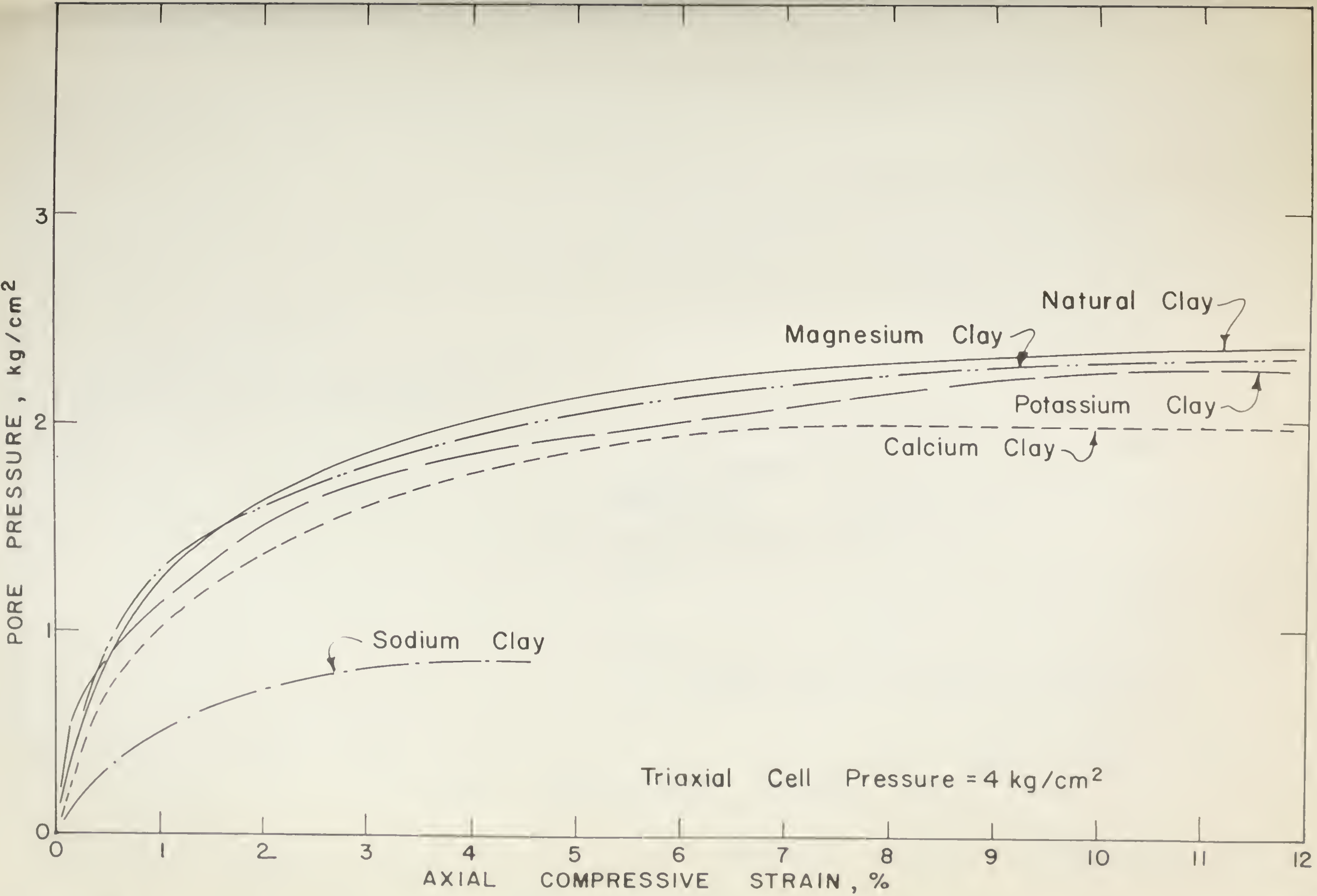

PLATE 6 PORE PRESSURE-STRAIN RELATIONSHIPS FOR THE CLAY MODIFICATIONS

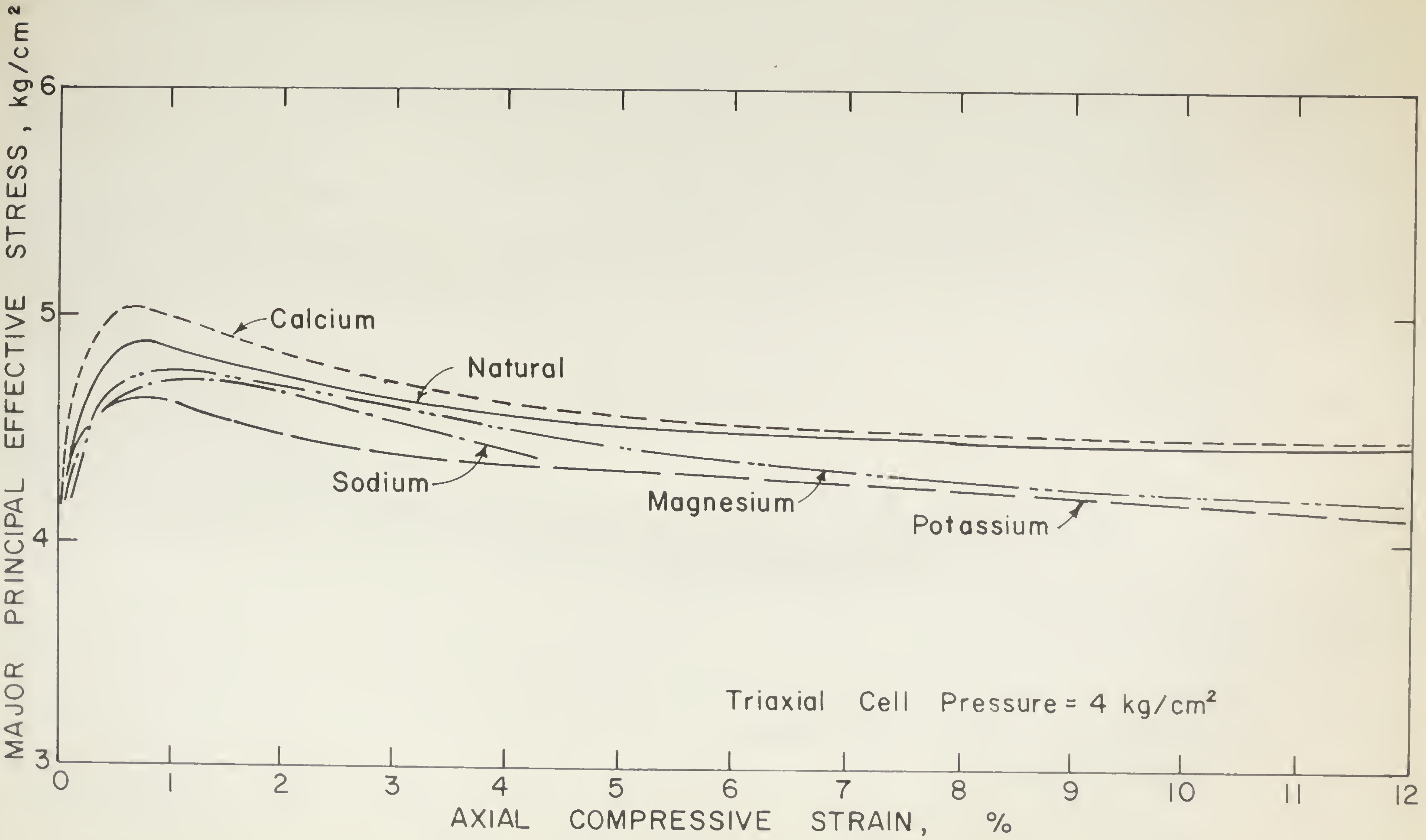

PLATE 7 MAJOR PRINCIPAL EFFECTIVE STRESS VERSUS COMPRESSIVE STRAIN FOR THE CLAY MODIFICATIONS

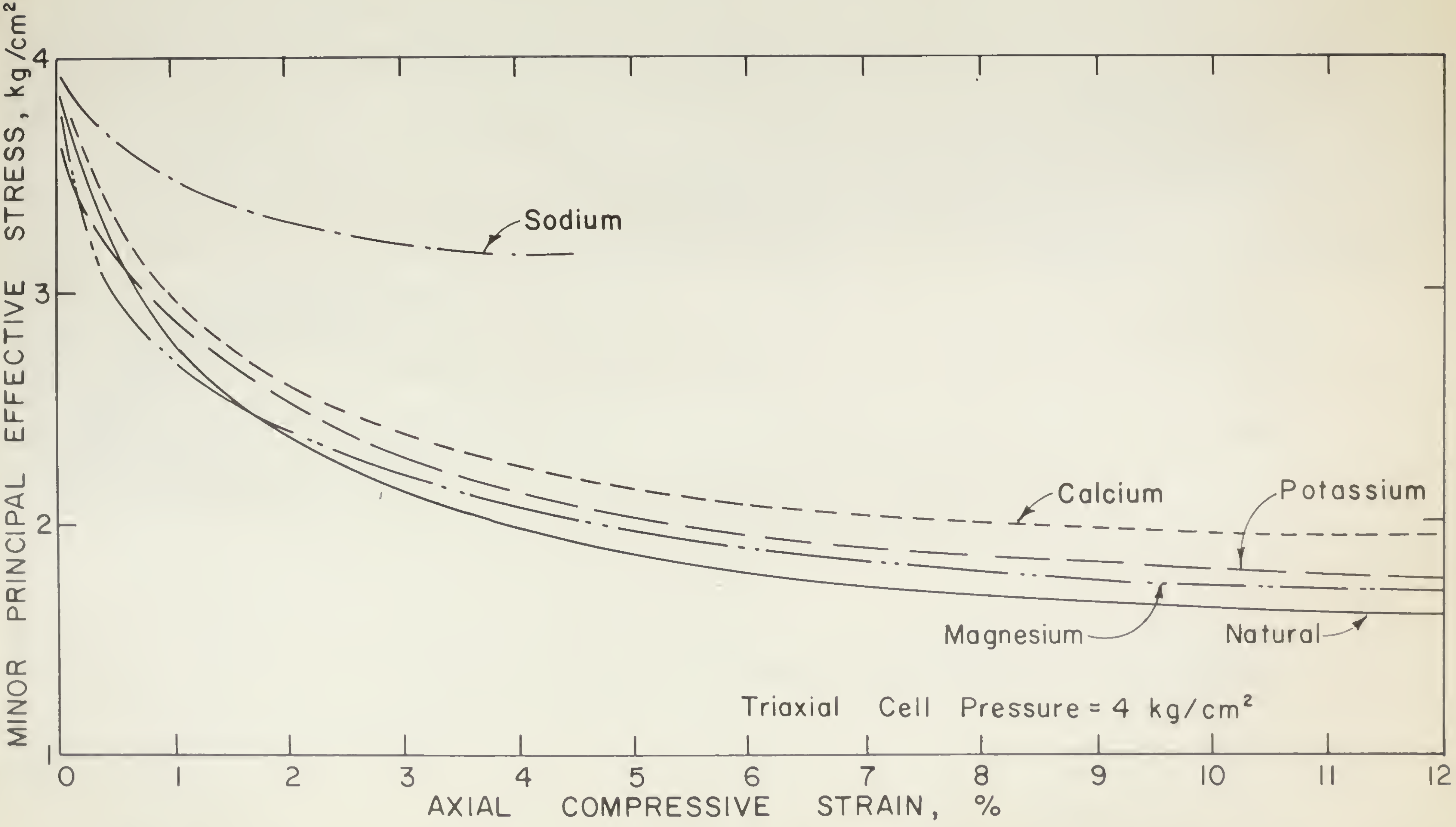

PLATE 8 MINOR PRINCIPAL EFFECTIVE STRESS VERSUS COMPRESSIVE STRAIN FOR THE CLAY MODIFICATIONS

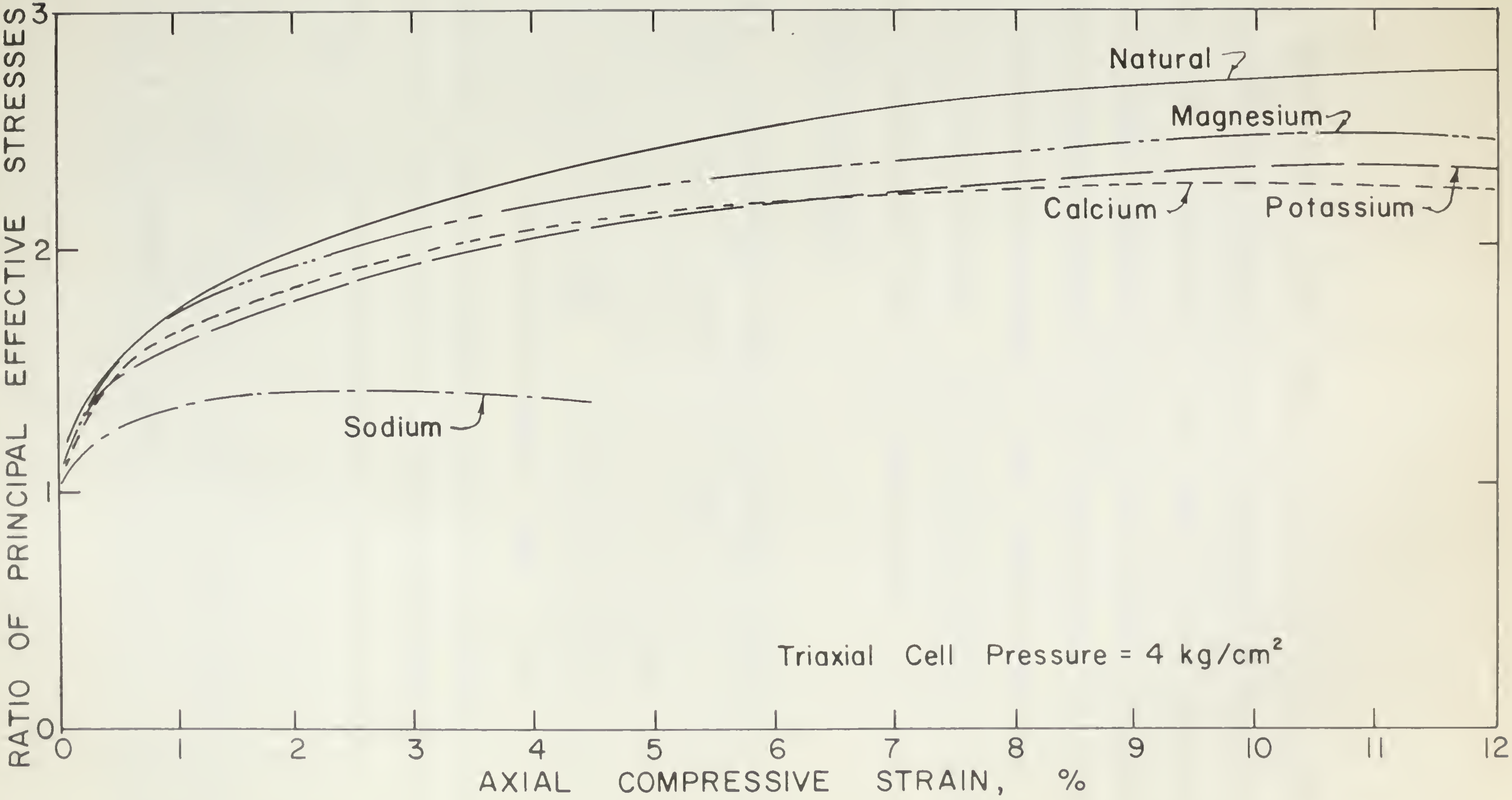

PLATE 9 RATIO OF PRINCIPAL EFFECTIVE STRESSES VERSUS COMPRESSIVE STRAIN FOR THE CLAY MODIFICATIONS

monovalent sodium ion and the interparticle forces are minimized. These effects are almost reversed for the divalent magnesium and calcium ions. A thin water hull is associated with these cations which then allows higher interparticle attractions and repulsions and, possibly, a tying of particles together due to the divalent nature of the cations. The potassium cation has a radius such that it can fit in the surface lattice layer of the clay minerals. Its co-ordination number is 12 which may be construed as the ability to knit sheets together. It thus has a higher strength than sodium.

5.20 The low deviator stress for the sodium samples may also be explained in terms of Lambe's equation (Para. 3.11). The high zeta potential associated with the sodium cation is responsible for a high R value, thus a low strength. For the divalent cations the zeta potential is lower, R is lower and the strength is higher. In the case of the potassium samples, the attractive forces, A, may increase due to the fact that the potassium ion can fit into the surface lattice of the clay mineral. An increase in A results in an increase in strength.

5.21 The strengths may also be considered in relation to the void ratio at failure. To illustrate, Table 7 has been compiled from Table 6 for a consolidation pressure of 4 kg per sq. cm. It will be

Table 7

Deviator Stress and Void Ratio for
the Soil Modifications Consolidated at
4 Kg per sq. cm.

Modification	Deviator Stress Kg/cm^2	Void Ratio
Magnesium	2.49	0.96
Calcium	2.47	1.03
Potassium	2.40	1.08
Sodium	1.37	1.56

noted that a decrease in the deviator stress is accompanied by an increase in the void ratio. Similar comparisons at the other cell pressures do not show the exact correspondence that is indicated in Table 7 but the same general trend is evident. The comparison of all soil modifications cannot be direct due to the variation in salt content among the samples.

5.22 The suggested explanation for the difference in the pore pressure developed in sodium as compared to the other modifications is developed by the following train of thought. To develop pressures in the pore water there must be a transfer of the applied stress to the water phase. This implies that a smaller proportion of the applied deviator stress (in the consolidated-undrained test) is transferred to the water phase in

the case of sodium than in the case of the other modifications. The low deviator stress shown on Plate 5 may also appear to be a reason for the low pore pressures associated with the sodium samples but the proportion that the pore pressure is of the deviator stress is lower in the case of sodium than in the other modifications. The ratio of pore pressure to deviator stress ranges from 60 to 75% for sodium samples and 85 to 95% for the other modifications. If it is assumed then, that there must be a stress transfer to the pore water then there must be a volume decrease of the soil skeleton at very low strains. In this context the soil skeleton is meant to include the adsorbed water hull. Such a volume change need only be very small, probably not enough to invalidate the constant volume concept associated with the consolidated-undrained test.

5.23 There would appear to be little doubt that such a small volume change is a distinct possibility and such changes were observed but unfortunately were neither observed nor recorded for all specimens. It is to be noted that the requirement is for the soil skeleton to decrease slightly in volume and not the entire sample, that is, there need not be a moisture content change. Elastic compression of the soil skeleton is one mechanism that could account for sufficient volume change to throw a greater stress onto the pore water. If one considers a sodium soil with its large water hulls then a disruption of the outer oriented water

is less dense than free water (22). Such a decrease in volume occupied by the water would tend to throw a greater stress on the soil skeleton. Since the oriented water was considered as part of the soil skeleton the decrease suggested may be interpreted as a decrease in the volume occupied by the soil skeleton. From the foregoing it follows that the high pore pressures require a soil skeleton that is capable of volume decrease whereas a proportionately low pore pressure demands a fairly rigid structure. The magnitude of the stresses involved must be borne in mind and hence such terms as 'rigid' are only relative.

5.24 If the foregoing arguments are valid, then from test observations of deviator stress and pore water pressure, the samples containing divalent ions (Ca and Mg) and sodium with high salt content in the pore water must have a soil skeleton that decreases slightly in volume while the sodium samples with very low salt contents have a 'rigid' structure. The following argument attempts to support this concept. Each clay particle is surrounded by a water hull, thick in the case of sodium, thin in the case of divalent ions and for high salt contents in the pore water. For thick water hulls a proportion of the hulls associated with two adjacent particles must overlap. Evidence in favour of such an overlap comes from the consolidation characteristics. It is proposed that the oriented water molecules within the overlapping water hulls impart

sufficient rigidity to prevent the transfer of applied stress to the pore water in the initial stages. This also allows for the low deviator stress encountered since the applied stresses need only break the bonding forces or disrupt the oriented water net which, at these large distances from the soil particle surface, are not likely to be very strong. This, as previously suggested, will also decrease the pore water pressures. The thin and well defined water hulls associated with the divalent calcium and magnesium cations do not overlap between particles. Therefore, when stress is applied the soil particles can shift slightly or slide one over the other sufficiently to bring about a slight volume decrease and to generate the high pore pressures.

5.25 The mechanisms presented in the preceding paragraph may also be used to account for the fact that the sodium samples fail at a low strain whereas the other modifications fail at a much higher strain. The large water hulls slide past each other with relative ease and, once the initial strain has disrupted the exterior portion of the water net, the water is now in a free state and the strength is further decreased. On the other hand for the divalent ions, the thin water hull is not disrupted and strength can continue to build up. The thin water hulls also do not 'smooth' out surface irregularities of the clay particles nor round out the particles hence more interparticle interference can occur again

leading to higher strains before failure occurs.

5.26 It may appear that the thin water hulls associated with calcium and magnesium should have the same bond between one another as do the thick water hulls. This does not seem to be the case due to the following considerations. Grim (20, 22, 24) reports that the transition of oriented water to free water is abrupt for the divalent calcium and magnesium cations. This sharp boundary suggests a strong orientation of the adsorbed water molecules and that hydrogen bonds do not act across this boundary. Particle reorientation that tends to align the particles in such a manner that basal surfaces become parallel or subparallel does not enhance strength due to bonding between water hulls when they are thin. These same considerations suggest that the internal friction is higher with thin water hulls.

Mohr Plots

5.27 During the plotting of the Mohr circles it became apparent that the six tests for each modification were defining two fairly distinct envelopes. A review of the results of the flame photometer analyses revealed that there were salts in the pore water. A test on a sodium modification with a deliberately high salt content in the pore water confirmed that the salts were a major influence on the test results. It was then decided to draw in the envelopes using the results of each batch of

three test results. Table 8 is a summary of the strength parameters. Full Mohr plots are in the appropriate appendix (Appendices F to J) from which the summary plots, Plates 10 to 14 inclusive, have been made. These plates are a somewhat modified Mohr Plot and were chosen for clarity of presentation of the data. Although somewhat impractical the angles of internal friction are reported to the nearest minute for discussion purposes. The cohesion parameter of Table 8 is largely of academic interest and represents the intercept of the Mohr envelope with the shear strength axis.

5.28 A study of the Mohr plots reveals two distinct facts. One that the adsorbed cation does affect the angle of internal friction to a greater or lesser extent. The other point is that the salt content of the pore or free water influences the strength of a given modification, the effect ranging from small for potassium soil to large for the sodium soil. These results must be considered more as qualitative than quantitative due to the uncontrolled salt contents which, however, do not invalidate the general conclusions.

Consideration of Angles of Internal Friction

5.29 For comparative purposes, the angles of internal friction for the various modifications have been plotted against the salt content on

TABLE 8

Summary of Strength Parameters

Modification	Salt Content of Pore Water me/100 gm soil	Total		Effective		Extrapolated Values for Zero Salt Content of Pore Water	
		c kg/cm^2	ϕ degrees	c' kg/cm^2	ϕ' degrees	ϕ	ϕ'
Natural	73.6 55.6	0.30 0.25	11°32' 10°32'	0.25 0.26	20°59' 19°24'	7°20'	14°27'
Potassium	31.8 5.1	.18 .24	11°27' 12°54'	.06 .27	22°50' 22°54'	13°09'	22°54'
Calcium	21.8 9.8	-- --	13°48' 13°36'	.05 .14	24°30' 23°30'	13°24'	22°42'
Sodium	-- 3.8	0.15 .05	6°25' 9°36'	.17 --	7°32' 13°18'	6°24'	7°30'
Magnesium	1.8 16.5	-- .09	13°36' 13°00	.05 .07	23°30' 22°42'	13°39	23°36'

Plate 15 and the resulting lines extrapolated to zero salt cont. It is felt that, for the modified soils, this extrapolation is an acceptable procedure since each modification has at least one fairly low salt content. For the natural soil, the extrapolation covers a large range of salt content and, therefore, may not be too reliable. However, due to lack of data at present, the results of the extrapolation have been accepted. The angles of friction thus obtained for zero salt content of the pore water are included in Table 8.

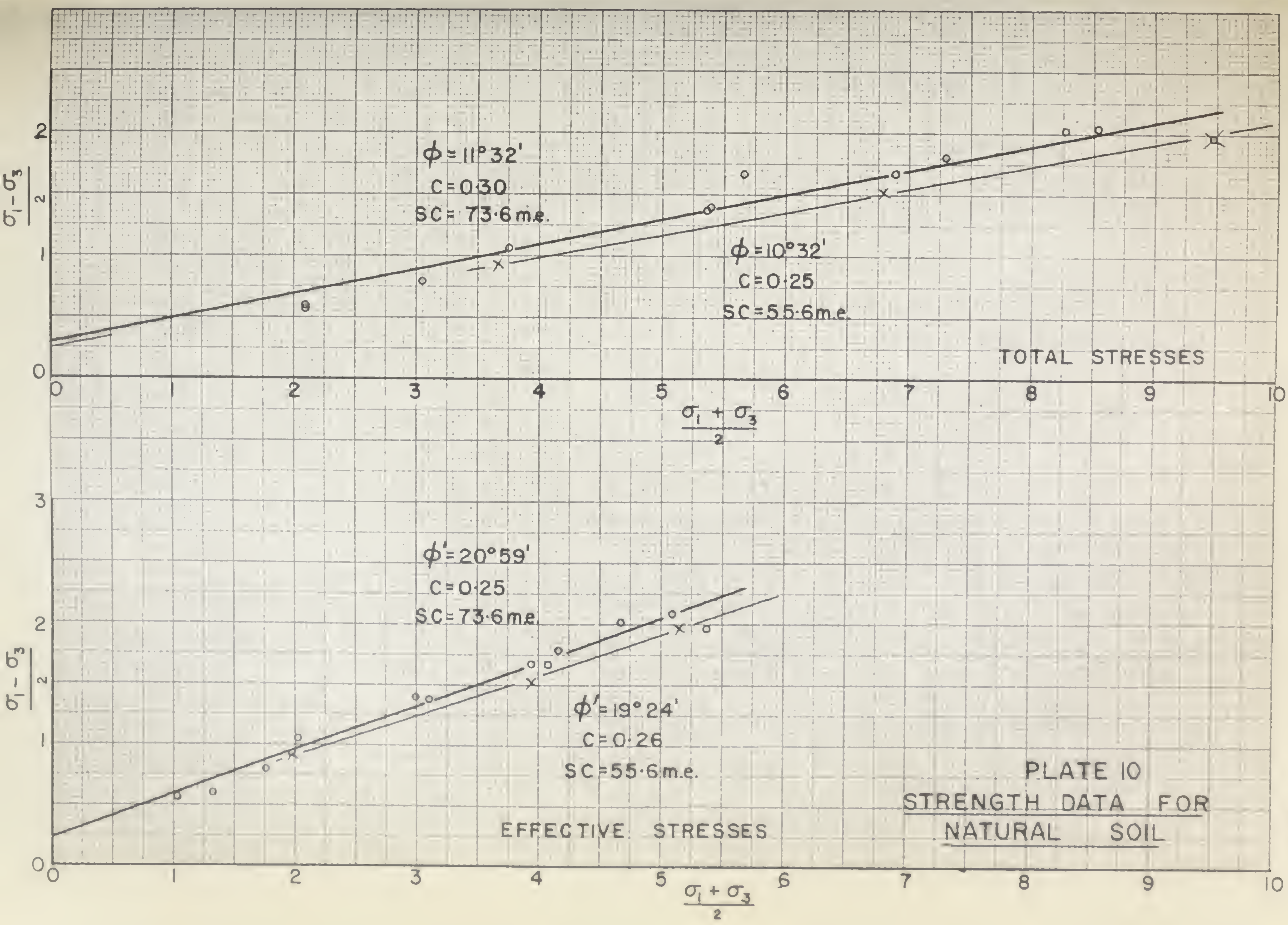

PLATE 10

STRENGTH DATA FOR NATURAL SOIL

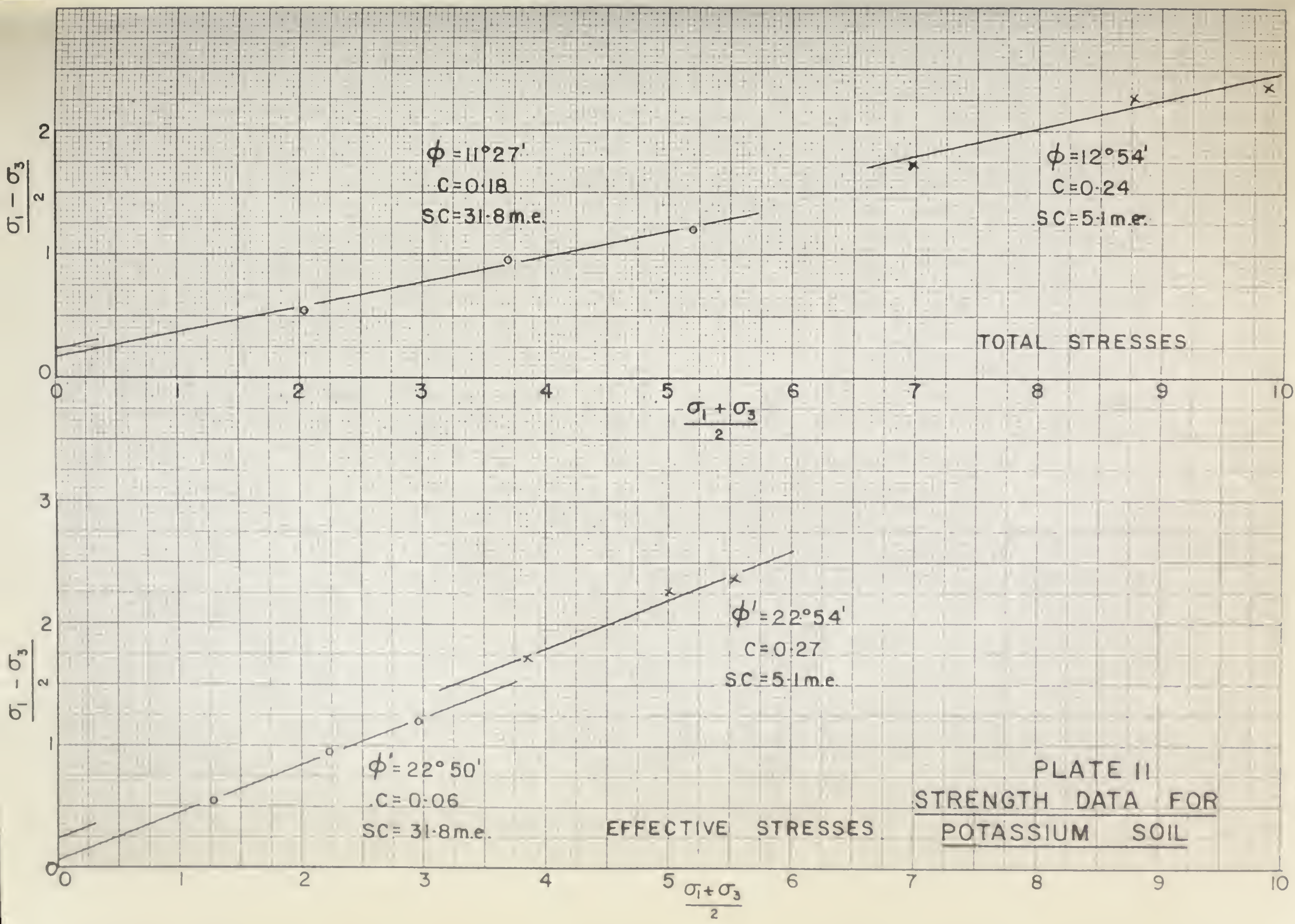

PLATE II

STRENGTH DATA FOR POTASSIUM SOIL

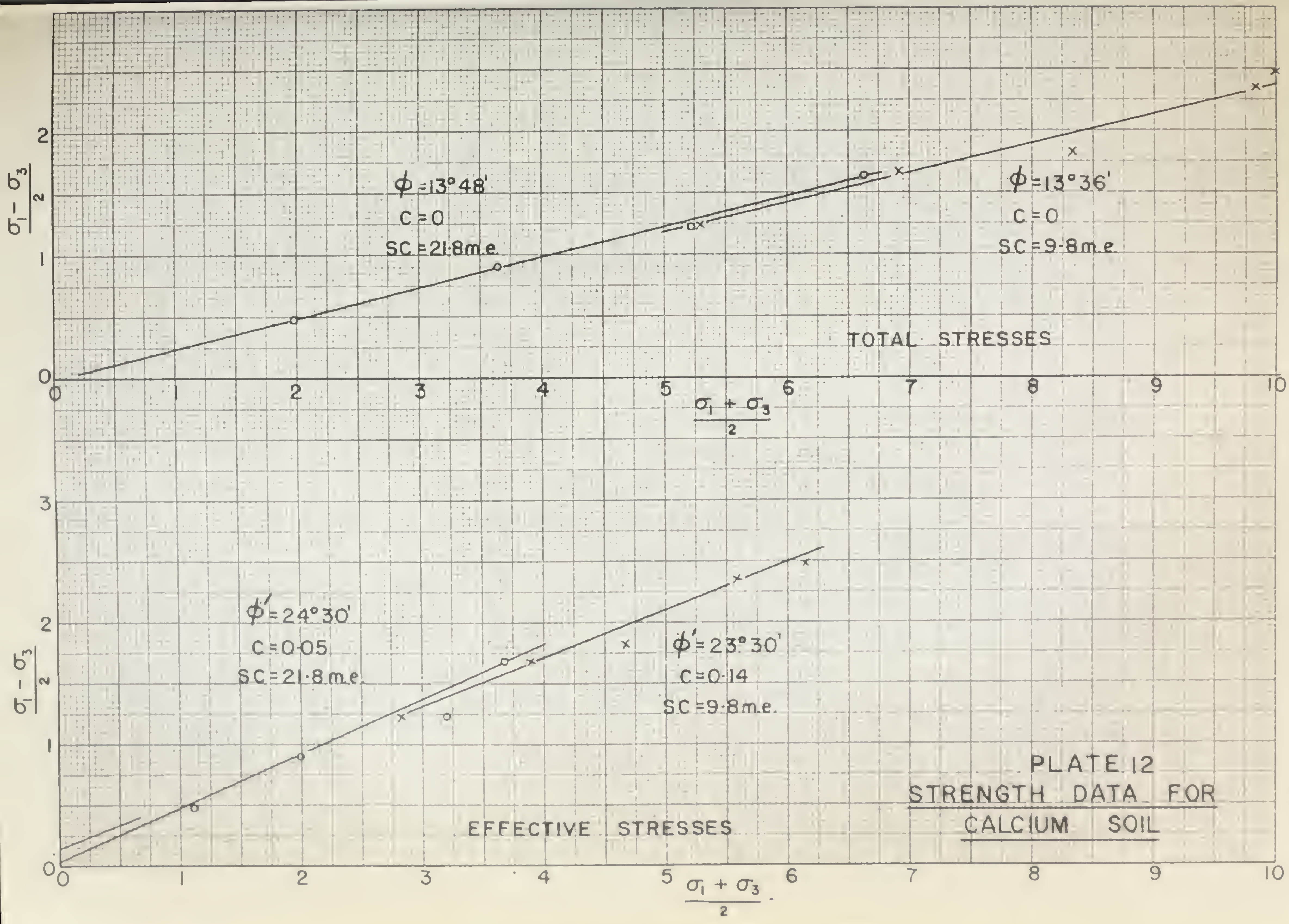

PLATE 12

STRENGTH DATA FOR CALCIUM SOIL

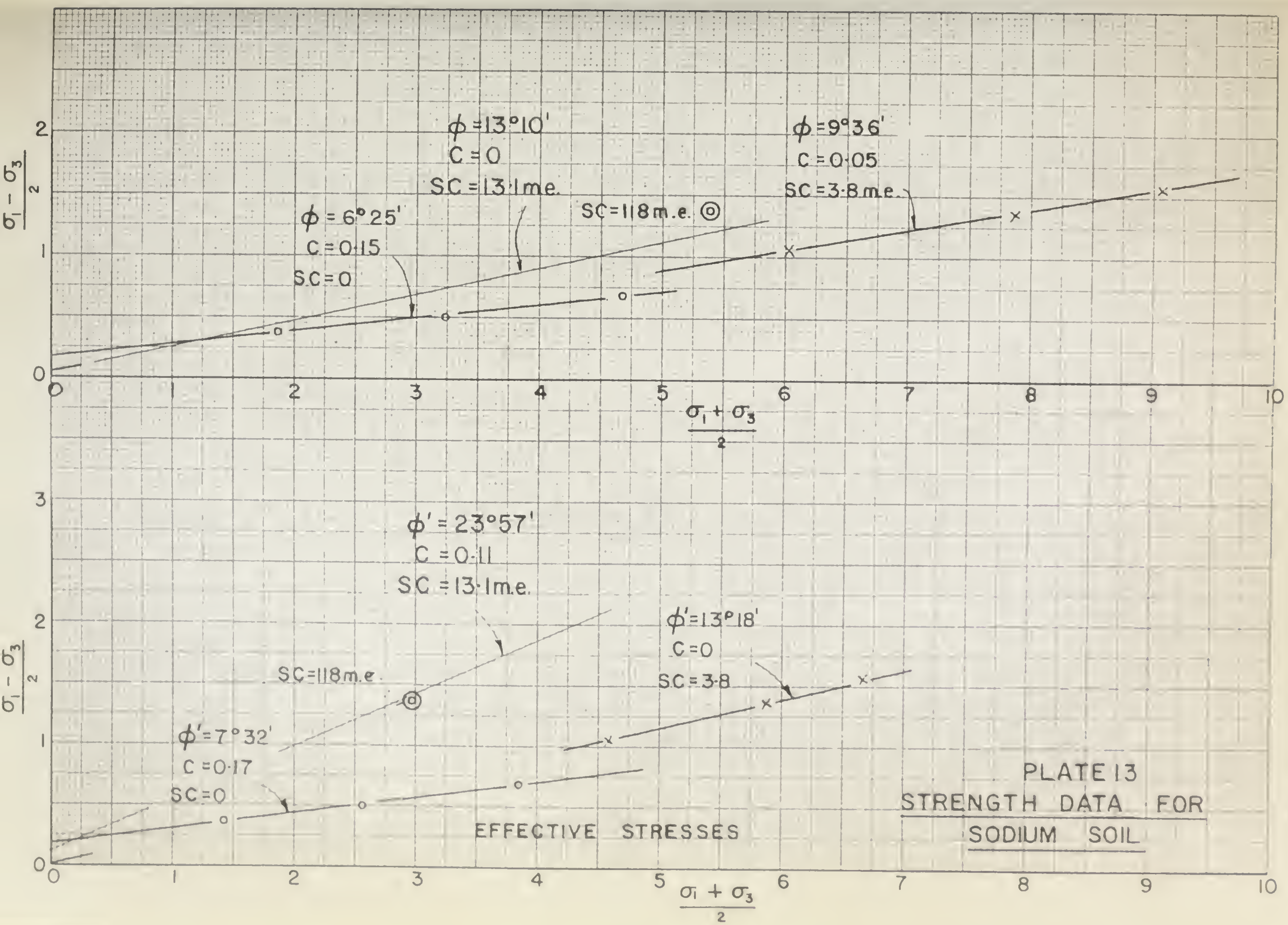

PLATE 13
STRENGTH DATA FOR
SODIUM SOIL

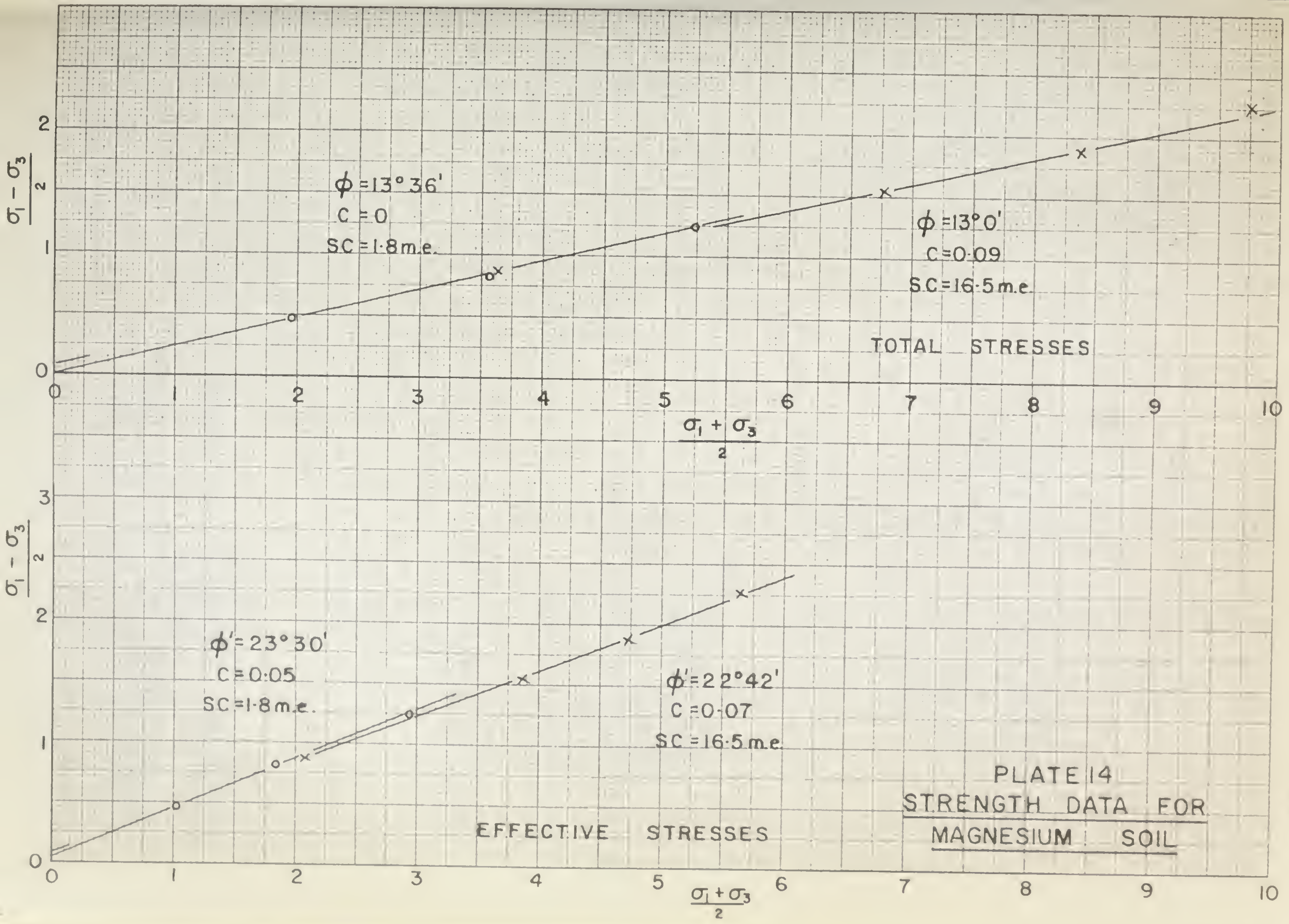

PLATE 14
STRENGTH DATA FOR
MAGNESIUM SOIL

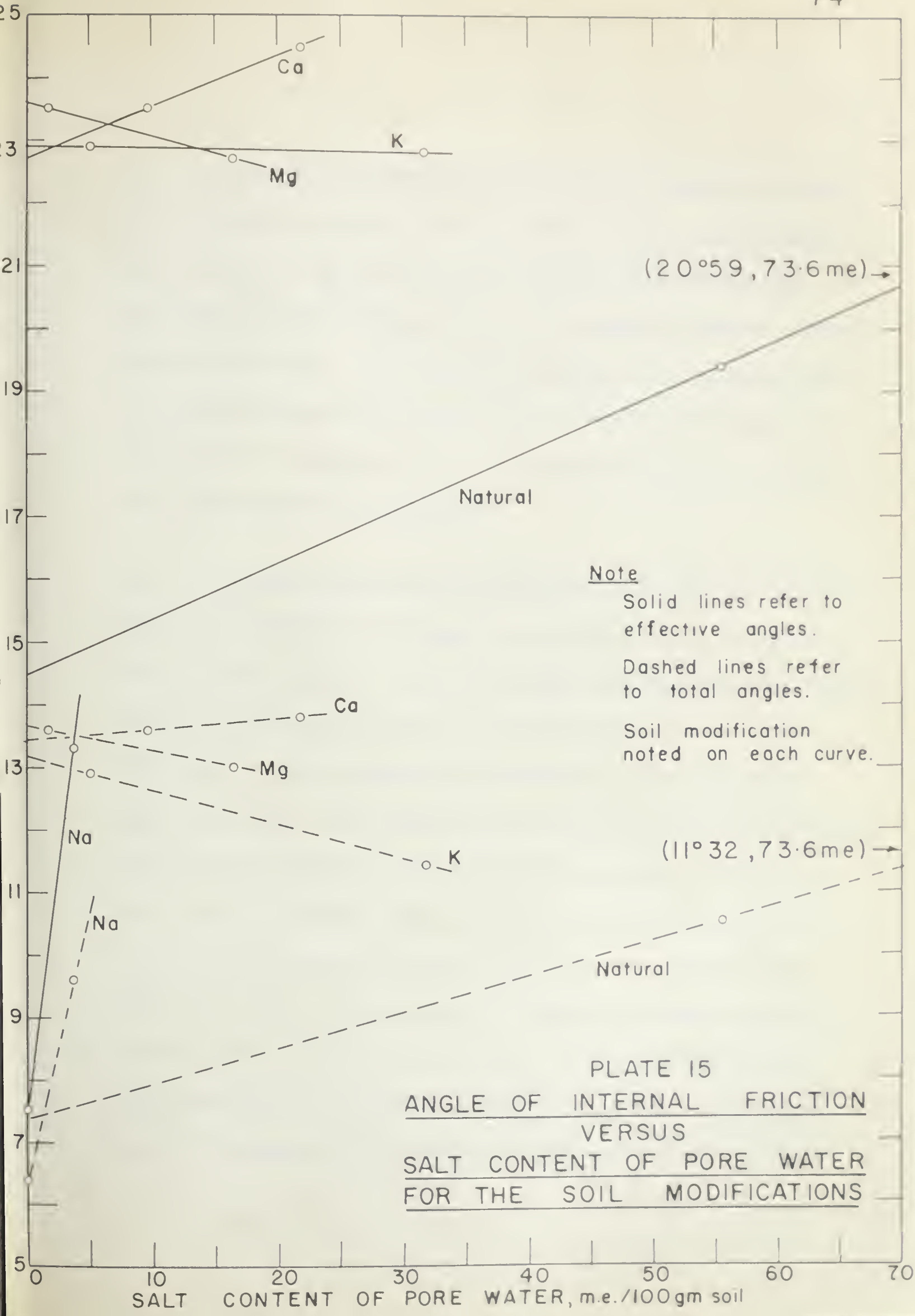

PLATE 15

ANGLE OF INTERNAL FRICTION VERSUS SALT CONTENT OF PORE WATER FOR THE SOIL MODIFICATIONS

5.30 The angles of internal friction at zero salt content for magnesium, potassium and calcium soils are about the same order of magnitude and all are considerably higher than that for the sodium soil. The arguments of the preceding paragraphs also appear to account for this difference in angles. The thin adsorbed water films associated with the divalent cations and potassium lead to maximum of interparticle interference by resulting in more soil per unit volume as is indicated by the void ratios of the specimens.

5.31 It has been reported by Bolt (6) and Grim (20) that the basal surfaces of clay particles are stepped as unit cells build on one another to form a complete particle. It can be readily visualized that these steps can cause interparticle friction as particles slide one over another. This type of friction is dependent on the normal stress and also on height of the step. This height will be an integral number of thicknesses of the unit cell which for montmorillonite is about 10A. A molecular layer of water is about 3A thick. In Paragraph D.14 of Appendix D it was stated that calcium soils have a water film about 15A thick and sodium about 100A. Friction between 'steps' of individual particles appears feasible in calcium soils whereas a water hull approaching 100A in thickness would mask or smooth out the steps. A further inference to be drawn from the water hull thicknesses is the type of

interparticle friction arising when two particles have their basal surfaces oriented at right angles to each other. This type of friction would also appear to be reduced by thick water hulls since the particles tend to be more rounded. In essence, the thin water hulls lead to more interparticle friction which is manifested in a higher angle of internal friction.

5.32 An anomaly that arises here is the fact that the natural soil has an angle of internal friction between the low value of the sodium soil and the high values of the other three modifications and has 84% of the exchangeable positions filled with calcium cations and 14% filled with magnesium. It has been suggested (70) that the presence of magnesium enhances the effects of adsorbed sodium and it is possible that the anomaly herein may be explained in a similar manner, that is, the intereffects of calcium and magnesium when both are adsorbed. In addition to this possibility is the fact that the anions in the natural soil are carbonate and sulphate radicals which also may have an effect. The general problem of investigating soil properties when more than one cation is adsorbed or when the anions vary will be left for future research.

5.33 The potassium soil, seems at first glance, to provide another anomaly since the potassium cation is monovalent but the modification

has a strength comparable to that of the soils of the divalent cations. The reason for this behaviour is the fact that the size of the potassium cation is such that it can fit into the hexagonal network of the basal surfaces of the clay minerals. This fit results in low repulsive forces and high attractive forces. All of these factors lead to high angles of internal friction.

5.34 The test results for the sodium modifications with varying salt contents appears to offer strong supporting evidence for the preceding postulates concerning the role played by the thickness of the adsorbed water film. On Plate 13 the strength data is presented with the salt contents in the pore water noted on the curves. The single test result for the specimen with high salt content of the pore water is plotted as is data obtained from other work.* The data observed during the compression testing of two sodium modifications, one having a low salt content and the other a high salt content, are plotted on Plates 16, 17 and 18. The affects of the salts are immediately apparent and behave as may be predicted by the previous arguments.

5.35 A significant point indicated by Plate 15 is the pronounced difference in the angle of internal friction for the sodium soil and those of

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* Data from G.L. Locker with permission by personal communication.

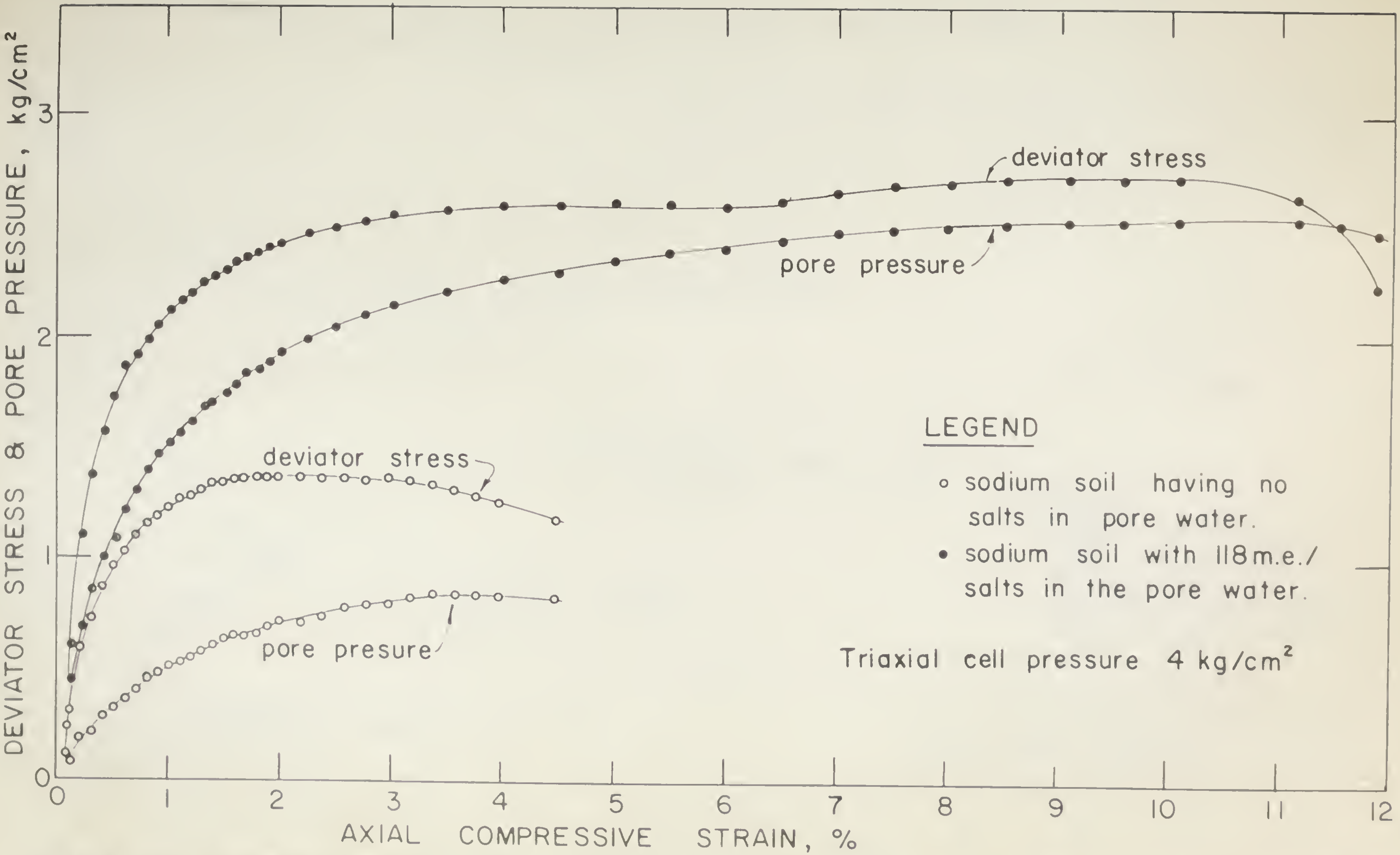


PLATE 16 DEVIATOR STRESS AND PORE PRESSURE VS AXIAL COMPRESSIVE STRAIN FOR SODIUM MODIFICATIONS

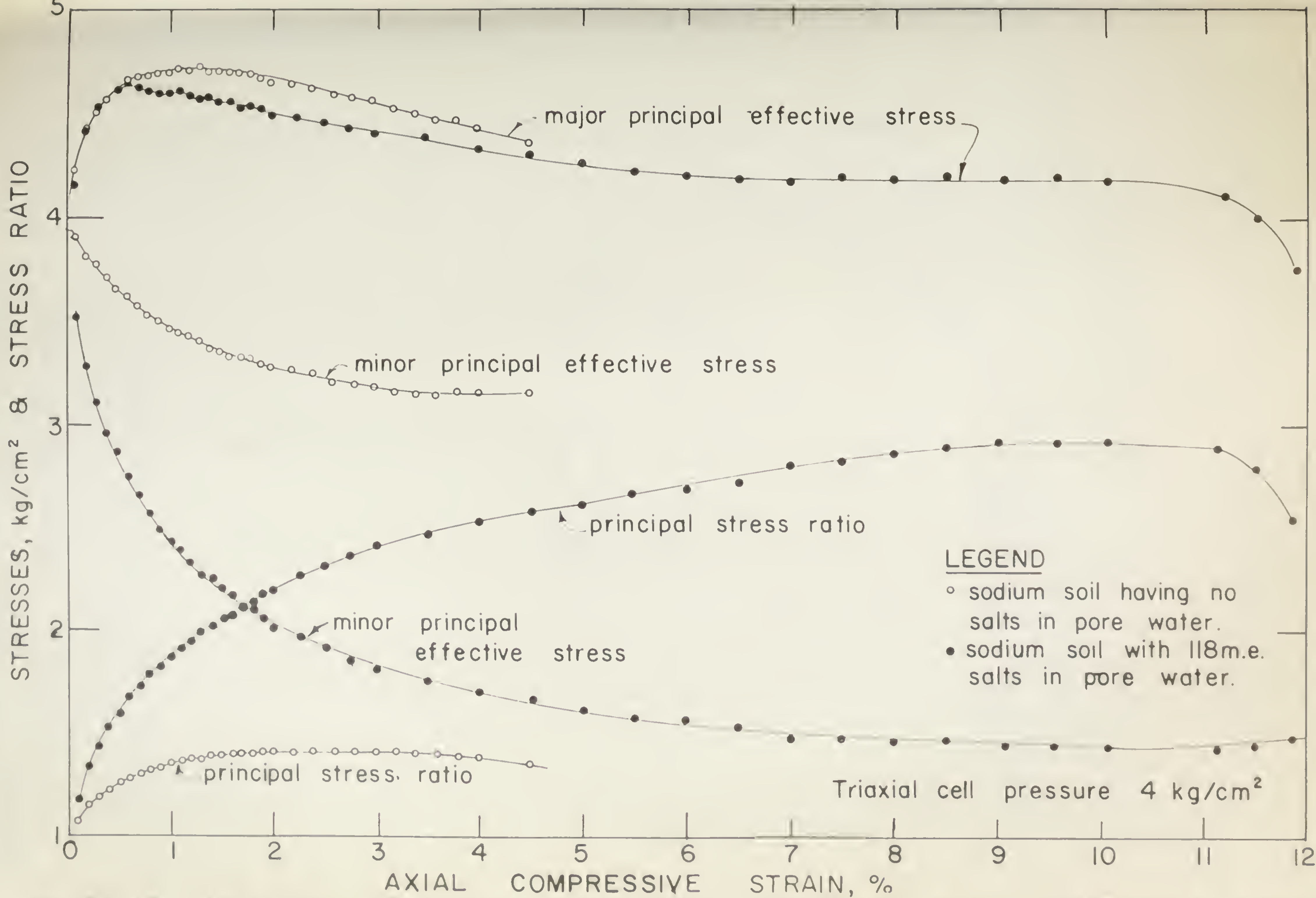


PLATE 17 PRINCIPAL STRESSES AND STRESS RATIO VS AXIAL COMPRESSIVE STRAIN

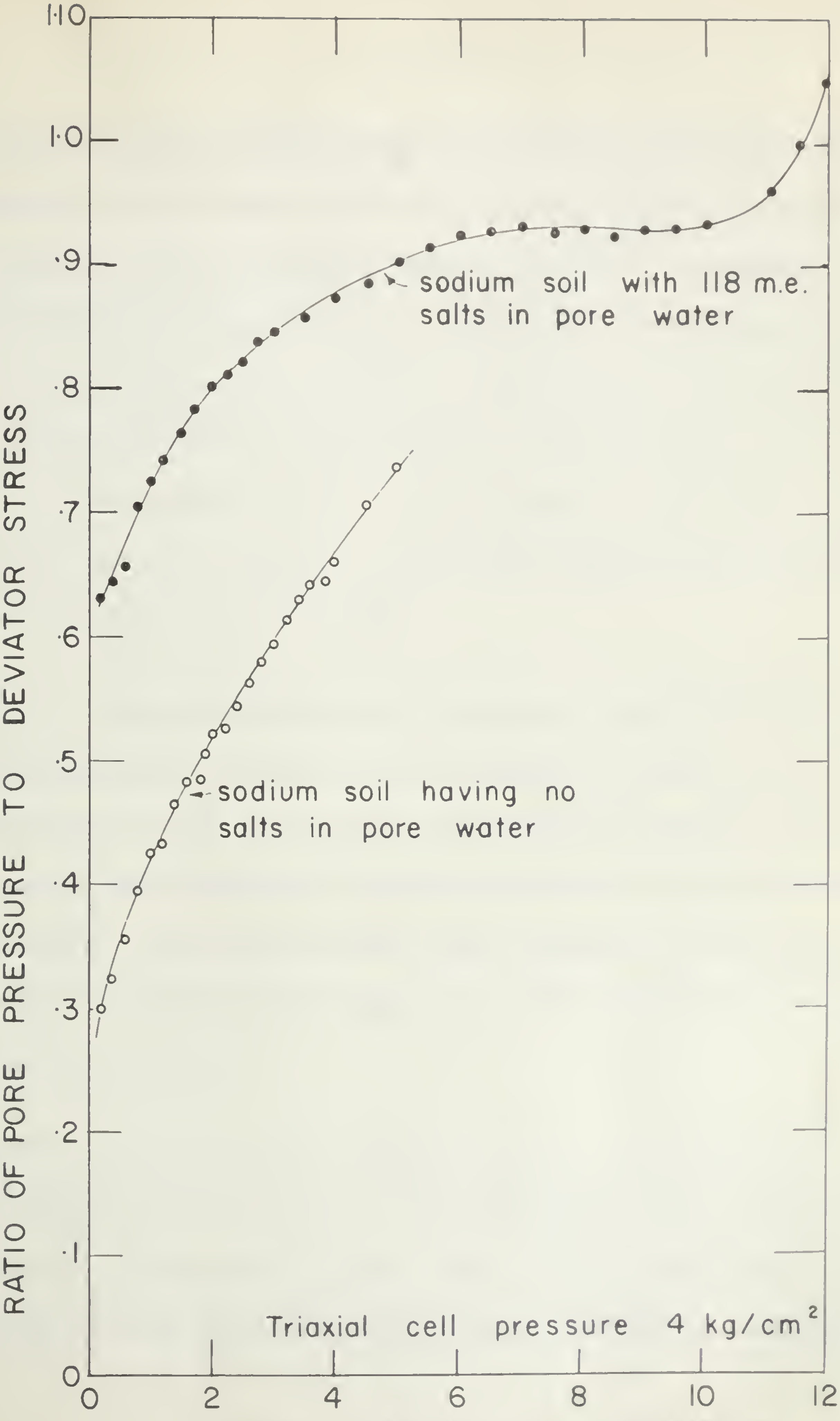


PLATE 18 RATIO OF PORE PRESSURE TO DEVIATOR STRESS VS AXIAL COMPRESSIVE STRAIN

the other cations. Magnesium, calcium and potassium soils have effective friction angles fairly closely grouped and in the order of 23 to 24 degrees; the corresponding angle for sodium is 7.5 degrees. If the salt content increases from zero, the strength of the sodium soil increases rapidly to the point where it is in the same order as the other modifications at the relatively low salt content of 13.1 m.e. per 100 gm of soil or about 2% by weight. This suggests that if a sodium soil exists in the field then a reduction in salt content below this 2% level will result in a drastic loss of strength.

5.36 A study of Plate 15 also shows the curves for the divalent cations and potassium to change slope with increases of the salt content of the pore water. The curve for the effective angle of internal friction for calcium rises while that of magnesium descends as the salt content increases. However, the actual angle changes are in the order of error of the determination of the angles hence it is not feasible to deal further with these changes.

5.37 The natural soil also appears to have a significantly lower angle of internal friction as the salt content of the pore water is reduced. However, as has previously been noted, the extrapolation may be in doubt, but if the strength losses are as indicated then it is suggested

that calcium-magnesium intereffects or the anions are responsible for the reduction of the friction angle.

Consideration of 'Cohesion'

5.38 A further study of Table 8 shows a tendency for the effective 'cohesion' to decrease as the salt content increases for potassium, calcium and sodium soils while the 'cohesion' of the natural soil and the magnesium modification remains essentially the same. There also appears to be a variation of cohesion with the adsorbed cation. Although these cohesion values are low, it is felt that they warrant some discussion. The term cohesion used in this context is the intercept of the Mohr rupture line and the shear axis. Cohesion has been variously considered as intrinsic pressure or interparticle attraction. The problem of developing an explanation for cohesion seems to be simplified by first postulating an interparticle 'glue' then considering the nature of this 'glue'. In the case of sodium clays the 'glue' appears to be hydrogen bonds acting at the edges or in the overlapping portions of the adsorbed water hulls. For the divalent cations (calcium and magnesium) the sharp boundary of the adsorbed film appears to preclude the hydrogen bond concept. The bond may be provided by the divalent cations which, due to their double charge exert an attractive force for adjacent particles. As the water hull thickness is decreased by salts in the pore

water the adsorbed cations are forced more closely to the negative surface of the particle which exerts more influence in satisfying the cationic charge. Also of probable significance is the fact that the presence of anions in the pore water tend to satisfy the cationic charges that would otherwise be directed to an adjacent particle.

5.39 The shift in the Mohr envelopes of potassium, calcium and magnesium apparent on Plates 11, 12 and 14 indicates a more marked change in cohesion than in the angle of internal friction. In each case the lower salt content represents the stronger modification. It would appear that variations in salt content also affect the shear characteristics by altering the cohesion. This is of obvious significance in a phi-zero analysis of the stability of a soil mass.

Summary

5.40 In summarizing this chapter, the explanations offered for the differences in geotechnical properties are based largely on the variations in thickness of the adsorbed water film. Grim (20) reports specific gravities of the clay minerals to be in the order of 2.6 when they are stripped of much of their adsorbed water. It is suggested that the adsorbed water film present on the clay particles influences the observed specific gravity normally determined in a soils laboratory.

The cation and the dissolved salts also exert an influence on the specific gravity. The adsorbed water film appears to play an insignificant part in the hydrometer analysis of fine grained soils. The increase in percentage of clay sizes is a result of cleavage taking place between the structural layers in the presence of sodium (20). The variation of the percent clay sizes with the zeta potential appears to substantiate the preceding statement since the greater the zeta potential the easier cleavage can take place. The time for consolidation to take place is also directly related to the thickness of the adsorbed water hull in the case of the sodium soil. It is suggested that the thick water hulls close off some pores completely and narrow down others to the point where there is considerably less space available through which water can flow. For the other modifications it appears that particle size is a major factor.

5.41 The variations in strength are also attributed to changes in thickness of the adsorbed water film. The thick films associated with the sodium ion tend to overlap which presents a rather stiff soil structure to initial imposed stresses. Due to this structure a larger proportion of the applied stress is carried by the soil skeleton of the sodium modification which is manifested in a lower pore water pressure. Once some strain has occurred, the outer fringes of the water film are

disrupted and more stress is thrown onto the soil skeleton. The thick water hulls are held responsible for the low strengths of the sodium samples. The reasons presented for this are that the thick hull smooths out the steps on clay particles and decreases effective particle interlocking. Also attributable to water film thickness is the variation in the number of actual soil particles that can be placed in a given volume. The effect is to create more or less interparticle friction as shearing strain takes place. The greater the thickness of the adsorbed water hull the less the angle of internal friction. The reverse is, of course, also true.

CHAPTER VI

GENERAL ASPECTS OF FINDINGS

6.1 In this chapter the implications of the findings of the research carried out and the ramifications of the postulates presented to explain the findings are viewed in the light of some current soil mechanics concepts. It must be borne in mind that the work undertaken for this report represents only a starting point from which many problems will evolve. It is hoped that further research will verify or modify the following views.

Surface Tension and Pore Water Salts

6.2 Dissolved salts change the value of the surface tension of water to varying degrees depending on the salt. For the common anions found in soil solutions (sulphates and carbonates) dilute solutions of one to two percent decrease the surface tension slightly from 74 to 73 dynes per centimeter while more concentrated solutions of ten to twenty percent raise it from 74 to 77 to 79 dynes per centimeter. These variations are relatively small hence would not appear to alter soil behaviour predicted on the basis of capillary forces. However, the presence of the salts

does decrease the adsorbed water film thickness, therefore, for a given void ratio the diameter of the capillaries will change. It is this factor that must be considered for shrinkage limit differences that may be observed in homionic soils with and without salts in the pore water.

Moisture Content versus Log Compressive Strength

6.3 It has generally been accepted that a plot of the moisture content versus the logarithm of the compressive strength results in a straight line for saturated cohesive soils. Plates 19 and 20 are plots of the data obtained in this program and very strongly suggest that the linear variation of moisture content with the logarithm of compressive strength only holds when the salt content does not change. This relationship is in line with the prior postulates concerning the role played by the salt content and its influence on the water hull of a soil particle. It was noted previously that cohesion appears to decrease with an increase in salt content thus forcing a soil-water system in the direction of a less cohesive mass. The fact that the slope of the lines on Plate 19 are flatter for higher salt contents substantiates this suggestion by indicating that the soil is assuming the characteristics of a less cohesive soil.

6.4 The behaviour of the soil as expressed by the plots of Plate 19, Moisture Content vs Log Compressive Strength, also seems to fit in to

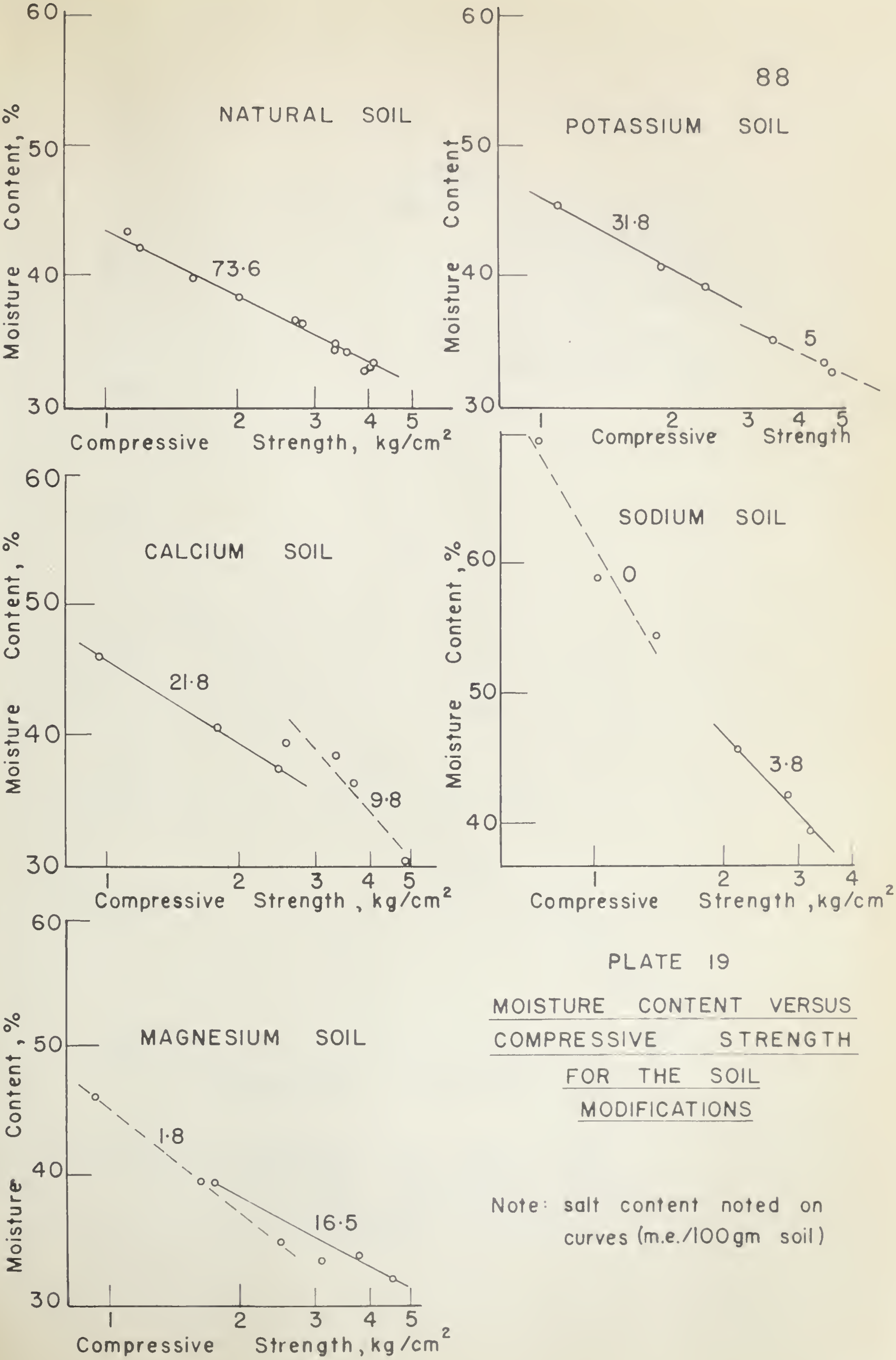


PLATE 19

MOISTURE CONTENT VERSUS COMPRESSIVE STRENGTH FOR THE SOIL MODIFICATIONS

Note: salt content noted on curves (m.e./100 gm soil)

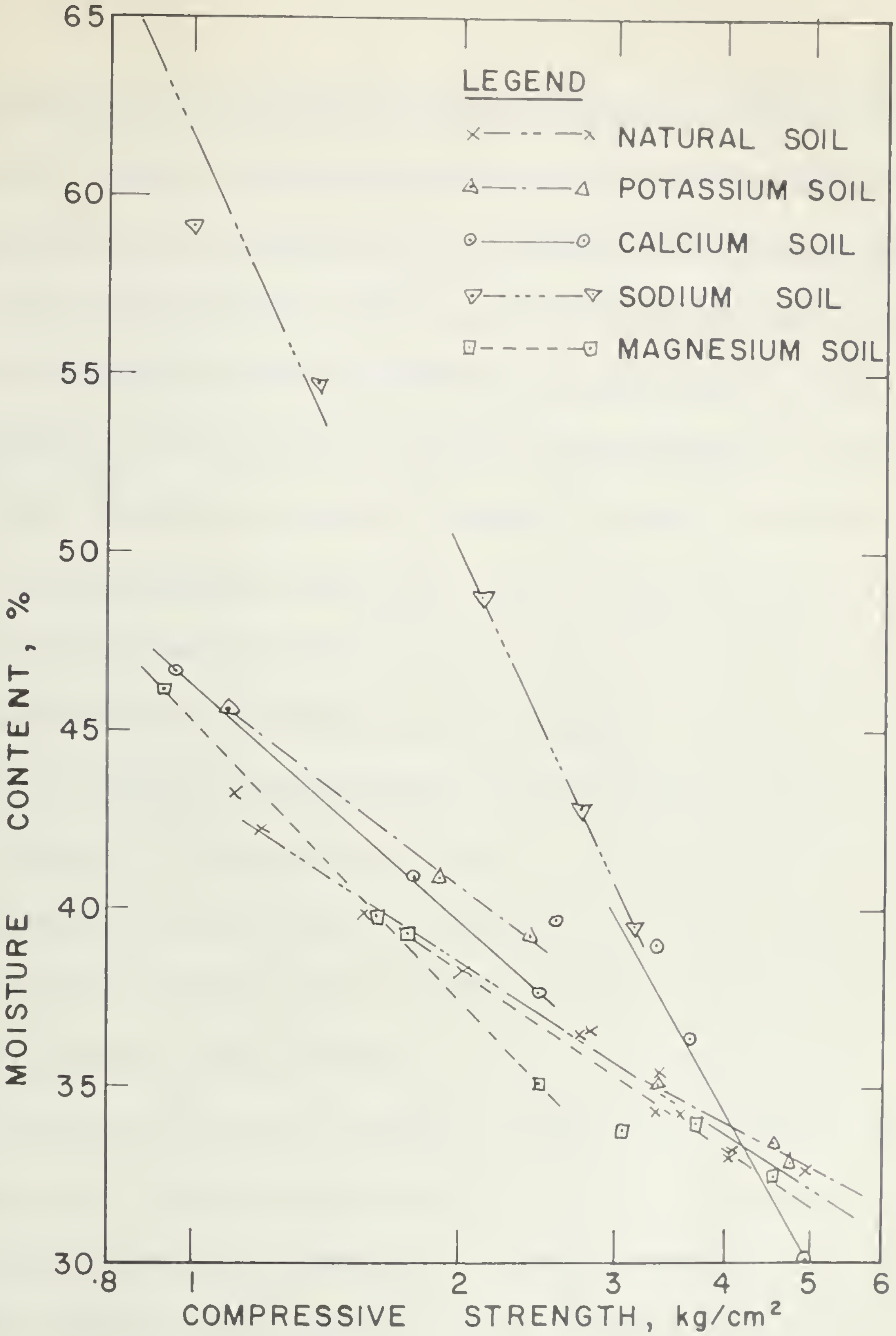


PLATE 20 SUMMARY OF DATA FOR MOISTURE CONTENT VERSUS COMPRESSIVE STRENGTH

the behaviour of clay soils when a non-polar fluid takes the place of water. When a symmetrical molecule such as carbon tetrachloride is mixed with a dry powdered clay a non-plastic mass results which has no cohesion (50, 51, 52). There appears to be no adsorbed film around the dry particle yet the adsorbed cations must still be present. The addition of a small amount of evenly distributed water to a dry clay powder results in a mass still lacking in cohesion. An air-dry powder is really composed of aggregates, each containing many particles. The cohesive forces for individual particles are satisfied by attachment to other particles within the same soil aggregate. Only when sufficient water is added can the aggregates be broken down thus freeing particles to take part in inter-aggregate bonds. The point here seems to be that the water film around each soil particle has to attain some certain thickness before the phenomena of cohesion begins to manifest itself. Grim (20, 24) reports this thickness to occur near the plastic limit. Within a certain small range of moisture contents in the vicinity of the plastic limit, the cohesion is dependent on the water film thickness, increasing as the film thickness increases. It would seem therefore, that within this range the reverse should be true, that is, decreasing the thickness decreases the cohesion.

6.5 From the preceding statements it is suggested that the adsorbed

water film plays an important role in cohesion. To reiterate what has been suggested earlier, a part of the cohesion for soils with thick water hulls may be due to the overlapping of these hulls and the hydrogen bonds built up within the overlapped portion. For thin water films with abrupt boundaries, interparticle attractions are postulated as the reason for cohesion. As well as Van der Waals forces there is the attraction of the positive cation swarm for the negative charge of an adjacent particle.

6.6 Plate 20 shows that, for a given moisture content (or void ratio) the sodium modification of the soil studied can have a strength greater than most of the other modifications. There is, however, a considerable variation in the confining pressures required to obtain a given moisture content, for example, the calcium modification having a salt content of 21.8 m.e. requires a confing pressure of 2 kg/cm^2 to yield a void ratio of 1.2 whereas the sodium modification having a salt content of 3.8 m.e. requires a confining pressure of 6.7 kg/cm^2 to yield the same void ratio (See Paragraph 6.28 and Plate 22). It may also be stated that for a given soil strength (for example 2 kg/cm^2) the sodium soil can have the highest moisture content. It must be inferred that the lower strengths of the modifications will obtain when volume increase is allowed. (See Paragraph 6.18).

6.7 It is also considered important to point out that all the curves on Plate 20 represent basically the same soil. All that occurred to result in the variations was the alteration of the salt content and the change of the ions occupying the exchange complex. It would appear, therefore, that a single plot such as Plate 20 cannot be relied upon for comparison of soils, particularly those containing a high percentage of montmorillonite, unless augmented by other data.

Variation of Cohesion and Internal Friction with Strain

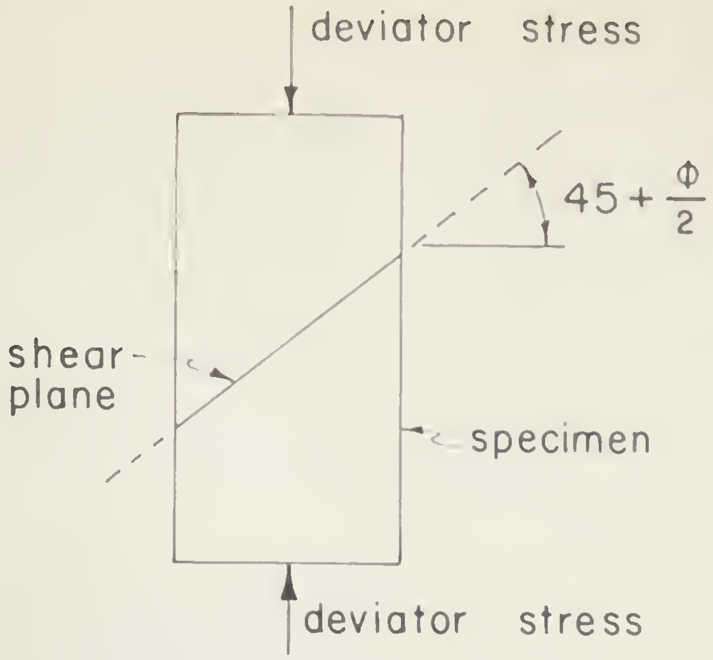
6.8 It would appear that the mechanism presented to account for cohesion and internal friction also may be used to account for the results of tests carried out by Schmertmann and Osterberg (56). These workers demonstrated that the cohesion is developed to its peak values at very low strains while the internal friction builds up much more slowly. The initial applied forces must first break the interparticle bonds which give rise to cohesion either by disrupting the outer fringes of the water films or by changing particle orientation which in turn means overcoming the established interparticle attractions. An initial relatively large force is required to achieve this since the system will have some inertia. Once overcome, the cohesion component of strength is small since continuing strain prevents re-establishment of the interparticle attractions. On the other hand, the internal friction is

building up since considerably more strain is required to develop interparticle interference to a maximum.

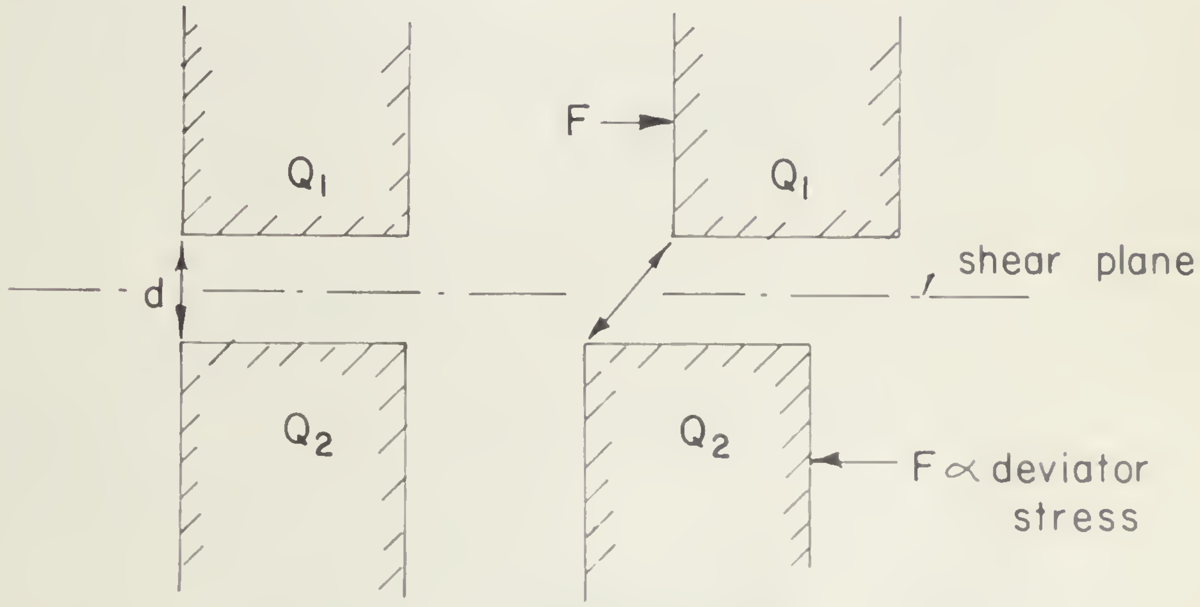
6.9 An interesting plot arises from the following considerations. Assuming that the shear strength of a soil is due to interparticle attractions and, in a simplified system, this attraction can be expressed by the Coulombic Equation

$$F \propto \frac{Q_1 \; Q_2}{d^2}$$

where F is the force developed between the charges Q_1 and Q_2 which are a distance 'd' apart. If these two charges are on opposite sides of a shear plane then the distance 'd' must be proportional to the strain and F must be proportional to the product of deviator stress and the sine of $(45 + \frac{\phi}{2})$. Plate 21 is a representation of this force field. For constant charges, a plot of the product of deviator stress and sine $(45 + \frac{\phi}{2})$ versus the logarithm of the strain should result in a straight line. Plate 22 shows such plots for typical results from the triaxial tests. For the upper three curves there is a straight line from about 0.3 to 5.0% or more strain. The deviation from the straight line at low strains may be due to instrumentation at these low values. Flattening out of the curves at strains greater than 5 to 8 per cent occur when the deviator stress nears its maximum value and begins to level off. Physically this would appear to be the point when internal friction



(i) General Relationships



(ii) Before Shearing

(iii) After Shearing

PLATE 21 SCHEMATIC REPRESENTATION OF A SIMPLE FORCE FIELD

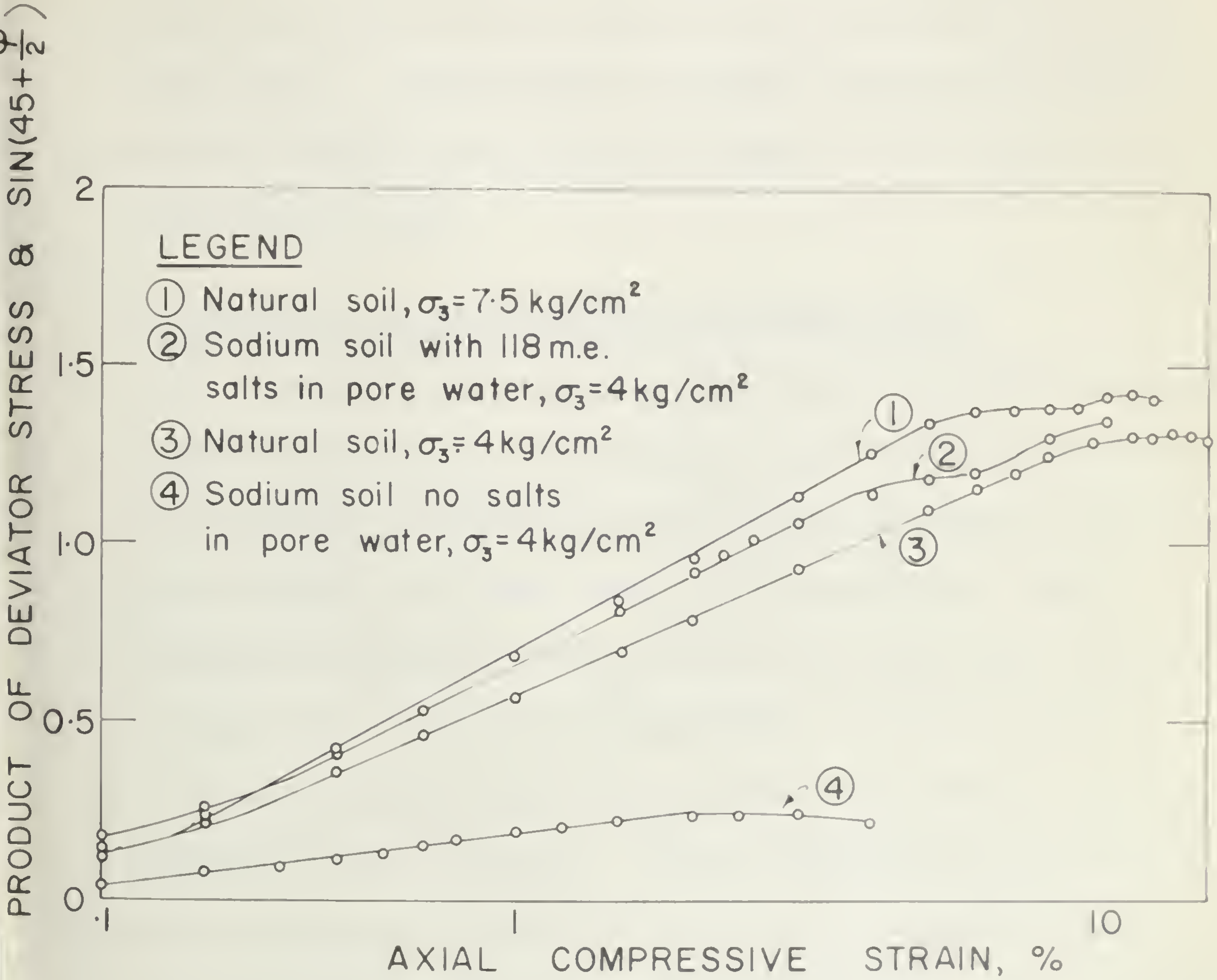
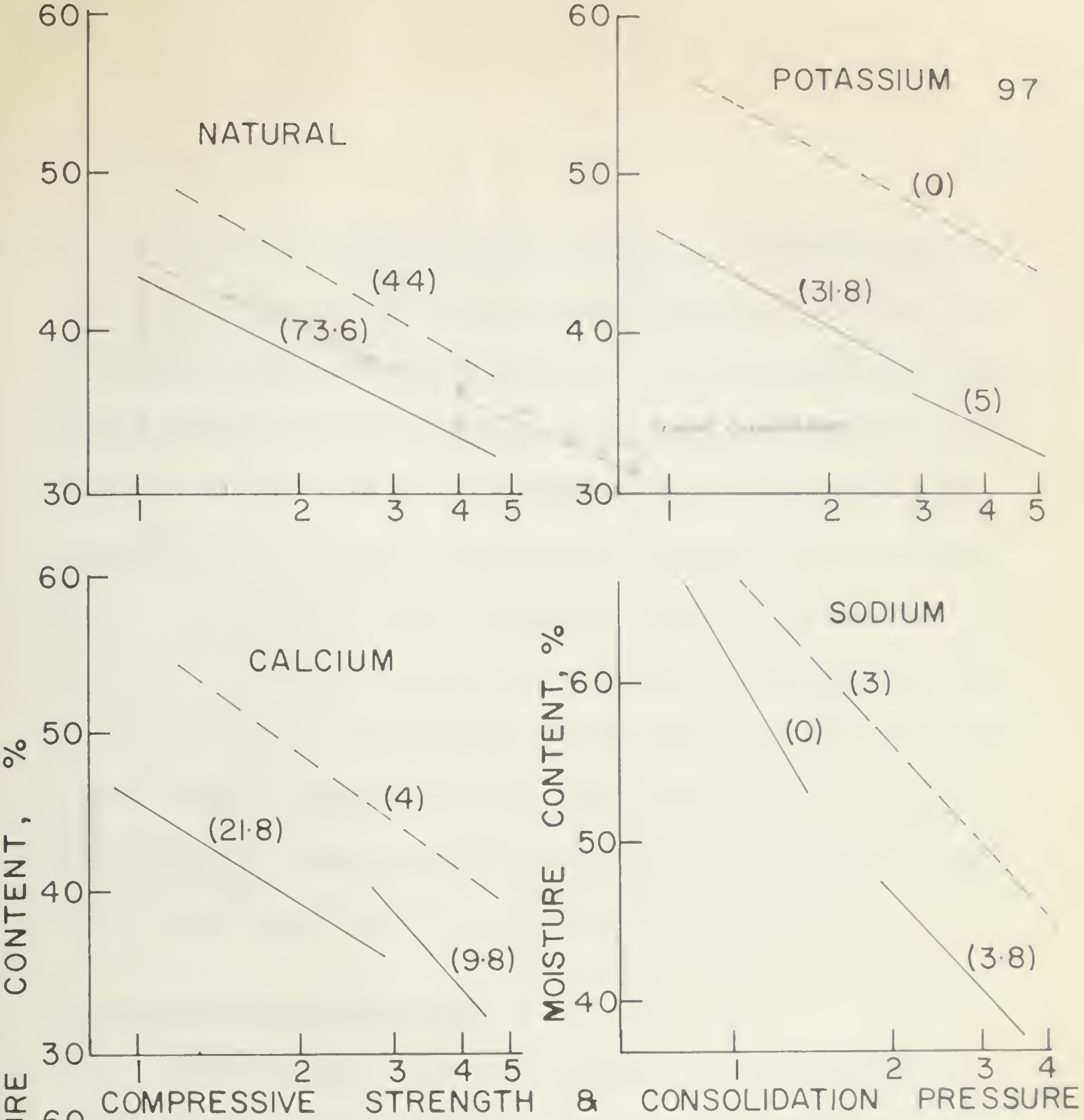


PLATE 22 PRODUCT OF DEVIATOR STRESS AND SIN(45+$\frac{\phi}{2}$) VERSUS AXIAL STRAIN FOR TYPICAL TEST RESULTS

has reached a maximum and a continuous shear plane has been established. Studies of the plots of Schmertmann and Osterberg (56) show peak friction angles to occur at about the same strain as maximum deviator stress. Plate 22 cannot be construed as proof of the validity of any of the postulates presented in this thesis but it does appear to lend some support to them.

Parallelism of Strength and Pressure-Void Ratio Curves

6.10 Taylor (61) and others discuss the concept applicable to saturated cohesive soils that a plot of the moisture content versus the logarithm of the compressive strength and the virgin compression branch of the pressure-void ratio curve should result in two parallel lines. Plate 23 presents the strength data derived herein and the consolidation data on the same soil from the work of Hamilton (26). In several instances there is a marked deviation from parallelism of the curves on each plot despite the paucity of the data. The major difference in the samples of each modification is the amount of salts in the pore water although the cation distribution on the clay complex may also differ. It would appear that parallelism of the curves should exist if the samples on which both tests are conducted are identical including the quantity of salts in the pore water. Consolidation may change the salt concentration as more water is squeezed out but this is unlikely to happen as long as the soil



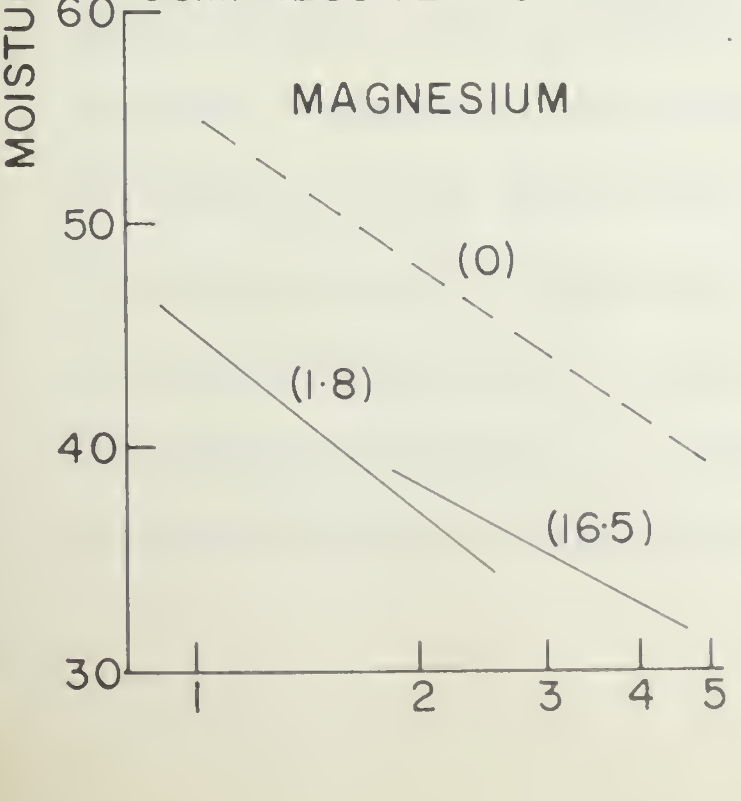


PLATE 23

RELATIONSHIP BETWEEN MOISTURE CONTENT, VIRGIN COMPRESSION BRANCH AND COMPRESSIVE STRENGTH OF THE SOIL MODIFICATIONS

Note:-pore water salt content noted on curves (m.e./100 gm soil)
-virgin compression branch dashed

is permeable to the ions of the dissolved salts. One would presume that as the difference in salt contents between the curves increases the divergence of the curves would increase. The plots do not particularly show this to be so except in the case of magnesium and perhaps sodium. The other two cations are contrary and in the calcium samples most noticeably so. Just what is the cause of this erratic behaviour is not known and more data would be necessary to resolve this situation. In any event Plate 23 does show that variations in salt content of the pore water do affect the relationship that exists between the strength curve and the virgin compression branch of the pressure-void ratio curve. The scattering of data encountered in other similar plots may, in part, be due to differences in salt concentration in the pore water.

Osmosis and Swelling Pressure

6.11 There have been postulates concerning the role played by the phenomenon of osmosis in swelling of soils and with regard to shear strength. This paragraph discusses this phenomena as visualized by the author as a result of this work. Osmosis is, by definition, a dilution of a concentrated solution by a solution of lesser concentration, the two solutions separated by a membrane. The term concentration is the number of 'particles' (e.g. cations or anions) in solution hence a compound that does not ionize will have a lower osmotic pressure than

one that ionizes in solution. General theory and some typical equations applicable in phsical chemistry are briefly covered by Maron and Prutton (41). The ions in a soil mass should be considered as two rather distinct entities, namely, those in the pore water and the cations adsorbed by the clay fraction.

6.12 The cations and anions existing in the free pore water must be considered as a source of osmotic pressure if a solution entering from an external source is at a lesser salt concentration than the pore water. The osmotic pressure visualized here is not to be confused with the pressure that may be generated when considering reactions involving adsorbed cations. In this case the 'membrane' separating the solutions must be taken as the soil mass itself with its capillary network, that is the external solution of low concentration can move into a soil mass but the pore water originally present cannot move out. Information concerning this point could be gained by consolidating a mixture of a soluble salt and a non-active colloid (e.g. sulphur) in a dry state in a consolidometer, then allowing water access to the system. The osmotic pressures developed should be approximated by existing equations. This general question is considered further in Paragraph 6.20.

6.13 The clay particle is surrounded by an adsorbed water film and

within this film are the adsorbed cations. This combination may be considered as a solution and, knowing the amount of the adsorbed water and the cation exchange capacity, the concentration of this solution may be calculated. It would appear that osmotic pressures may then be calculated from a knowledge of the concentration of the pore water. If it is assumed that the plastic limit of the soil mass represents the amount of water necessary to satisfy the hydration demands of the clay particles, then this figure combined with the cation exchange capacity yields a concentration which may be used to calculate osmotic pressures. The results of this calculation yield osmotic pressures far in excess of any ever measured. The reason for this lack of agreement lies in the fact that the preceding calculation assumes that all the adsorbed cations are free to enter into an osmotic reaction. This assumption is not true due to the simple fact that the cations are attracted to the particle surface. A factor is required by which calculated osmotic pressures can be multiplied to yield more realistic results. Alternatively, such a factor may be applied to reduce the concentration of the solution involving the adsorbed cations, that is, one must determine what proportion of the adsorbed cations are free to enter osmotic reactions. The determination of such factors is the realm of the physical chemist or soil scientist rather than the engineer.

Adsorbed Water Films and Swelling Pressure

6.14 For the purposes of the engineer a simpler explanation is to postulate that swelling pressures are largely a result of increases in thickness of the adsorbed water film. Grim (24), in discussing thixotropy, discusses the reorientation of water molecules and states that it is a crystalization process. The large pressures built up when confined water freezes are well known and these pressures are due to crystalization. It would seem logical to postulate the same mechanism when water molecules are reoriented by the force field of a clay particle. Oriented adsorbed water is less dense than free water and in occupying a larger volume must exert a swelling pressure. The word 'largely' used in the first sentence of this paragraph is meant to imply that there are sources of swelling pressure other than the water film postulate. Osmotic pressure, as mentioned in the previous paragraphs, and elastic rebound of the soil are other possibilities.

6.15 Desiccation of a soil mass or deposition of salts due to downward leaching of water will increase the salt content which results in a thinning of the adsorbed water films thus, under consolidation loads, a denser state can be achieved. If a soil mass is assumed to be in a dense state with a reasonably high level of salts in the pore water and water becomes available to the mass then, based on the preceding

discussions, the following will take place. First assuming the available water is pure (e.g. rain or meltwater), osmotic pressures will build up due to the difference in the concentrations of the pore and external water. The dilution of the pore solution will allow the adsorbed water film to expand which, in turn, will tend to exert a swelling pressure. Both of these mechanisms will continue until an equilibrium is established. This equilibrium must consist of two entities, first an osmotic equilibrium keeping in mind that as swelling goes on a diffusion of the solutions is taking place and second the equilibrium between salt content of the pore water and the zeta potential. If the water available contains dissolved salts the osmotic pressure will be lower and there will be a reduced swelling pressure.

6.16 It would appear that if a sufficiently concentrated solution of salts were available to the soil then no swelling should occur and even in some instances an actual decrease in volume could occur. In the case of highly colloidal soils, the event of no swelling seems unlikely to occur due to the fact that a saturated solution of some salts can be maintained in the soil due to some having been precipitated out during the past history of the deposit. The implication here is that an added solution will be less concentrated than that present in the soil. Volume decrease would not occur due to the resistance offered by the soil

skeleton. The fact that swelling pressures can develop that are in excess of their overburden pressure is not an anomaly in the light of the postulate of swelling pressures being in equilibrium with salts in the pore water.

6.17 Based on increasing water film thicknesses, it is possible to estimate theoretical amounts of swell. Assuming a saturated clay soil to have a moisture content of 50% and the specific gravity of soil solids to be 2.8, then the total height of a volume of (1 + e) will be 2.4 units. It has been suggested (20) that adsorbed water may have a density of 0.7. Accepting this figure, and assuming that the amount of water adsorbed increases from a small amount to 50% of that present in the sample, the height of a volume of (1 + e) now becomes 2.7 units. This represents an increase in height of about 12%.

Considerations of Field Problems

6.18 The concentration of the salts in the pore water in a soil mass in the field becomes an important factor in the light of the explanations presented in this report. Referring back to the Mohr plots, Plates 10 to 14 and Table 7, it is shown that the adsorbed cation influences the angle of internal friction. These data also show the marked influence on strength that is exerted by the level of the salt concentration in the

pore water. The condition that exists may be termed an equilibrium of the soil with its immediate environment. Any disturbance of the equilibrium conditions will lead to a new equilibrium under the new conditions. Therefore, a soil mass in the field should be considered as dynamic system rather than as a static one. If one imagines water percolating through a soil mass several possibilities may be considered assuming the overall salt content to remain essentially constant. If the water had previously been through a limestone area and, as a consequence, had calcium cations in solution these would effectively replace any adsorbed sodium and result in a calcium soil. If, on the other hand, the leaching water held only sodium cations, the soil mass would slowly assume sodium characteristics with lower strengths and a capacity for higher swelling pressures. There is a problem of the replaceability of calcium by sodium since the former is more tightly adsorbed than the latter. However, continued leaching with a sodium-rich solution would bring about a change. If the percolating water contained both calcium and sodium cations, the soil would preferentially adsorb calcium (33) and assume the associated characteristics.

6.19 If consideration is given to a change in the salt content of a soil mass there are two general possibilities, viz., salt level decrease or salt level increase. The most pronounced effect of salt-level decrease

is illustrated on Plate 13, the Mohr diagram for the sodium test results. In this particular instance, the effective angle of internal friction was reduced from about 30 degrees to 14 degrees when the salt level in the pore water was reduced from a very high level (118 m.d./100 gm) to a low level (3.8 m.d./100 gm). Assuming zero cohesion, this represents a strength loss of 57%. Pure water or zero salts results in a further reduction in the angle of internal friction to 7.5 degrees or a total strength loss of 77%. The other cations, potassium, calcium and magnesium, show very little change in the angle of internal friction but the envelopes for the potassium and calcium shift downward, that is a decrease in cohesion, as the salt content increases. Appendix L, Results of Tests on Washed Natural Soil, also reveal some strength loss as the salt level of the pore water of the natural soil are decreased. One would presume that the reverse of these phenomena would occur if the salt content were increased.

6.20 One point of immense significance is whether or not any volume change can accompany changes in the salt content of the pore water. A study of Plate 19, Moisture Content versus Compressive Strength, shows that the samples of each modification having the lower salt concentration possess the higher strength for a given moisture content or void ratio. Also, as was observed from Plate 20 and in Paragraph 6.6,

replacement of magnesium by sodium leads to strength increases if the void ratio remains constant. These statements infer that, if no volume increase can take place, a lowering of the salt content or adsorption of the sodium ion will increase the compressive strength of the soil. On the other hand, if the soil mass swells strength losses will occur as the salt content of the free water decreases or as sodium is adsorbed. Which of these alternatives will take place will depend on field conditions. It would appear that if man-made structures are the means of confining the soil they may be subjected to considerable swelling pressures.

6.21 The disturbing of the equilibrium that has been suggested as existing in a soil mass in the field (Paragraph 6.18) may also have serious consequences. If, as would appear to be the case, many natural slopes exist whose factor of safety against failure is close to unity then it would only require a small change of the equilibrium conditions to precipitate a land slide. What would seem to be minor amounts of construction, for example a shallow side hill cut, may be enough to change the ground water regime by leaching of the ground water salts or by decreasing the overburden pressure. Either of these factors would allow the adsorbed water films to increase and decrease the strength of the soil mass sufficiently to start a slide. The basic problem is the determination of the equilibrium that may exist in field.

The solution to this problem will involve considerably more research.

6.22 A fact of some significance is that the very low strengths of a sodium soil occur only at extremely low levels of salts in the pore water and that as these increase in concentration the strength increases. The point at issue becomes the level of concentration of salts in the pore water. If these are high and remain so then no strength decrease is possible from this phenomena. If cation exchange is to occur (i.e. adsorption of sodium) then the salt content must be presumed to be high enough to prevent strength loss. The solubility of the salt which provides the cation is also of consequence. In general, sodium salts are more soluble than are those of calcium hence sodium salts are more likely to be removed by leaching before those of calcium. A salt such as calcium sulfate is reasonably insoluble but still enough so to provide a continuing source of calcium cations. An observation in favour of a general high salt content in the pore water is the presence of much selenite ($CaSO_4 \cdot 2H_2O$) in the clay soils of many slide areas. Many of the soils in these slide areas also have a marked reaction with dilute hydrochloric acid which denotes a high carbonate content.

6.23 There are instances of muskegs or organic terrain located above slide areas that are a source of water seeping into the sliding zone. In addition to increasing the moisture content and providing seepage

pressures, water from muskegs may contain organic dispersants (57). These organic compounds behave in a manner similar to sodium and impart to the soil similar and undesirable properties. Corrective measures may include replacement of the organic dispersants with a cation such as calcium as a procedure that can be put into effect in a short time at a low cost pending other corrective techniques such as drainage.

6.24 The cations in the immobile layer are thought to be dehydrated due to being so strongly adsorbed (See Paragraph D.17 of Appendix D). If it is assumed that sodium ions exist in this immobile layer and that they are slowly replaced by the processes of weathering, there would appear to be a possibility of some strength decrease due to the release of sodium cations and their subsequent effect on the water hull thickness. Since weathering normally takes place at shallow depths, the infiltrating water is likely to be reasonably pure rain water. In this event the sodium will not be immediately exchanged and its influence on soil properties may involve some time.

Salt Concentrations in Ground Water

6.25 Field problems may involve the salt content of the pore water therefore it is pertinent to consider briefly the ground water of the

Province of Alberta. The following information was provided by Mr. C. E. Noble, Alberta Provincial Analyst, during a recent discussion of the subject with the author. Analysis of the wells in a belt along the mountains shows the water to contain considerable hardness in the form of magnesium and calcium salts. The next broad division is a rather wide belt flanking either side of the Calgary-Edmonton Highway. The water of shallow wells (up to 90 feet deep) has hardnesses ranging from 350 to 1500 parts per million (ppm) and in addition contains Glauber's Salts ($Na_2SO_4 . 10H_2O$) in amounts varying from 100 to 4000 ppm. The latter salt is often concentrated in pockets and sloughs. In terms of sodium sulfate, the range of 100 to 4000 ppm corresponds to concentrations of 0.0014N to .056N or 1.4 to 56 m.e./l. The general area to the east, centered roughly by Vegreville, contains NaCl at depths in the vicinity of 60 to 90 feet. Wells deeper than about 100 feet contain $NaHCO_3$, NaCl and Na_2SO_4 at an average of 1500 ppm. In very general terms, the salt content of the ground water will contain the salts of magnesium, calcium and sodium in the range of 5 to 70 m.e./l. There will undoubtedly be local variations even perhaps over short distances. According to Kelley (33) the divalent cations will be adsorbed in preference to sodium, a fact that accounts for the high sodium content of the pore water.

6.26 Even the preceding brief resume of the ground water salt contents is sufficient to point out to the soils engineer the necessity of generating an interest in measuring the level of salt concentration and, perhaps, determining the cations and anions present. The upper level of the salt content mentioned is in excess of those of the modifications tested hence some of the results of this work have direct field applications. It may be pertinent in a field problem to visualize the changes that could occur if the salt content changes during the life of the structure. To illustrate this point consider a secondary, unpaved road constructed on a clay subgrade. If the original salt content is high sufficient strength is available to result in adequate service. However, as rain and meltwater leach the salt out, the strength decreases and the road becomes impassable.

6.27 It is interesting to calculate the osmotic pressure arising when distilled water dilutes concentrations of solutions as reported in the preceding paragraph. If a hardness of 350ppm and a Glauber's salt content of 4000 ppm is assumed then the concentration of the solution is approximately 0.06N. In terms of sodium, this equivalent to 0.06 moles per litre. At a temperature of $25^{o}C$ the following simple formula gives:

$$\Pi V = nRT$$

$$\Pi \times 1 = .06 \times .082 \times 298$$

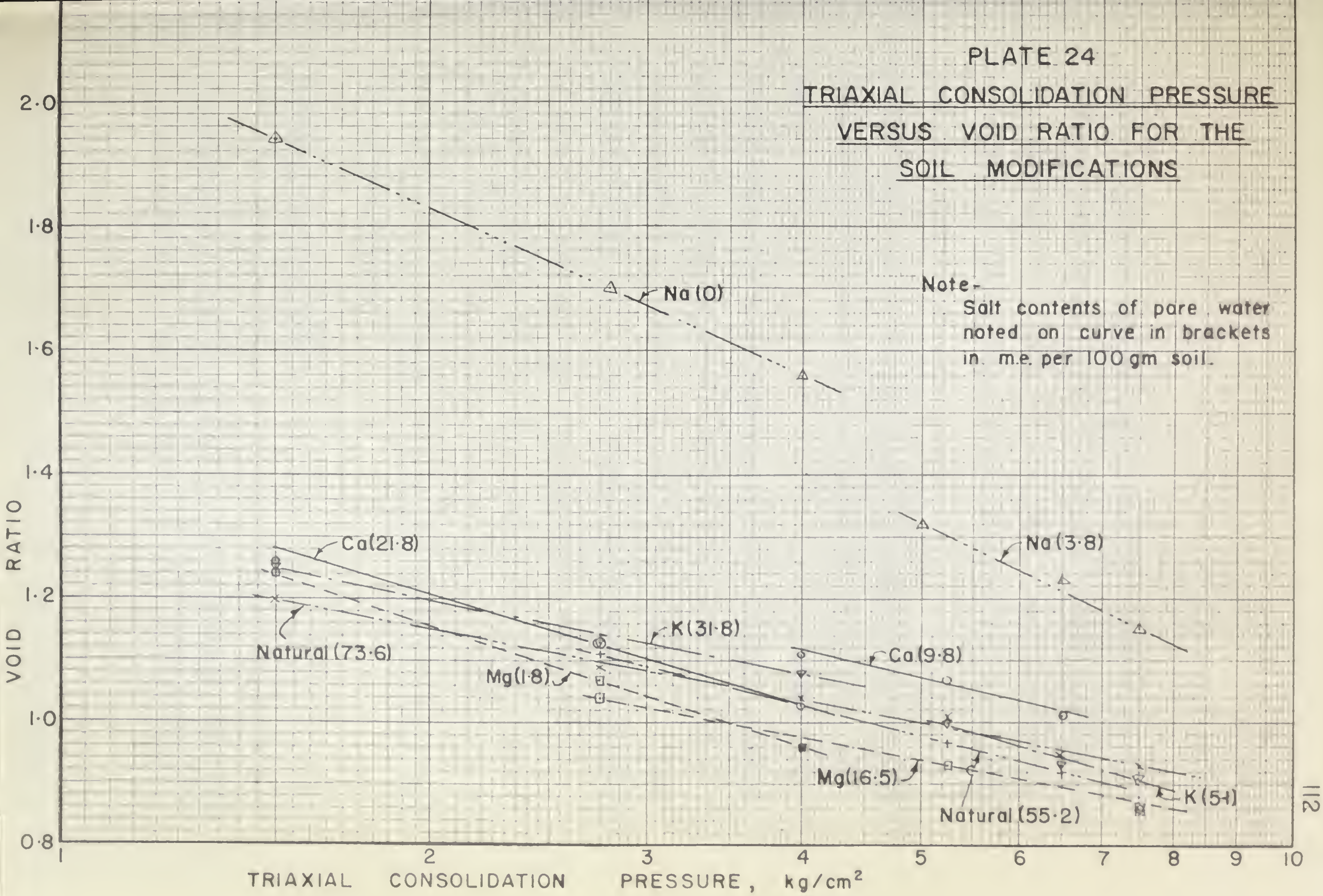
$$= 1.55 \text{ tons per sq. ft. (kg per sq. cm)}$$

where Π is the osmotic pressure developed by a solution of 'n' moles in a volume 'V' at an absolute temperature 'T'. The gas constant is 'R' and the units are so chosen to give dimensional stability. This figure is of the same order of magnitude as the swelling pressures that have been measured on may soils of Western Canada.

6.28 At pressures normally encountered in soil mechanics, one would expect that the void ratio at the end of consolidation would reflect the influence of the adsorbed cations and the salt content of the pore water. On Plate 24 this void ratio has been plotted against the consolidation (cell) pressure and it is readily apparent that the void ratio is, in fact, dependent on both the adsorbed cation and the concentration of salt in the pore water. This plot also indicates the cell pressures required to yield a given void ratio. If the cell pressures are obtained from this plate for a void ratio of 1.0, it will be noted that an increase in cell pressure is required as the salt content decreases for the calcium, sodium and magnesium soils. This may be explained in terms of the increase in osmotic pressure as the salt content of the pore water decreases. As the osmotic pressure increases so must the consolidation

PLATE 24

TRIAXIAL CONSOLIDATION PRESSURE VERSUS VOID RATIO FOR THE SOIL MODIFICATIONS



Note -
Salt contents of pare water noted on curve in brackets in m.e. per 100 gm soil.

pressure increase to achieve a given void ratio. However, for the potassium and natural soils, for decreasing salt contents a decrease in the cell pressure will yield a void ratio of 1.0. Since the potassium cation fits into the clay mineral surface, these cations are not free to enter into osmotic reactions hence interparticle attractive forces are predominant. The attractive forces act in the same sense as the consolidating pressure thus reducing the cell pressure required. To explain the behaviour of the natural soil, one must again turn to the intereffects of calcium and magnesium cations. These data suggest that the interaction of these two cations is such that their activity is reduced which in turn reduces the resulting osmotic pressure. For higher void ratios, there is more volume available and the intereffects of calcium and magnesium would be reduced thus a higher osmotic pressure would result which, in turn, would require a higher consolidation pressure. This would explain the fact that at a void ratio of 1.1, the natural soil sample having the lower salt content requires the higher consolidating pressure.

Consideration of Soil Types

6.29 The physical-chemical phenomena dealt with in this thesis are a manifestation of the surface acitivity of the soil particles. The cation exchange capacity of a soil is, in some respects, a measure of this

surface activity. Rock flour or fine silt which possesses a low or negligible exchange capacity would be expected to show practically no strength variations with either the adsorbed cation or salt content variations. On the other hand, a soil containing large amounts of montmorillonite, for example bentonitic clays, would reveal large variations in geotechnical properties with changes in the adsorbed cation or salt concentrations in the pore water. The general point is that the concepts outlined in this thesis are applicable to fine grained soils containing appreciable quantities of the clay minerals, particularly montmorillonite. A qualitative measure of the magnitude of the changes in the physical characteristics of a soil with variations of the salt content or adsorbed cations would appear to be the cation exchange capacity. As the cation exchange capacity is high, thus denoting a high surface activity, the magnitude of the changes would be expected to be large. Determinations of which ions are adsorbed and in what proportions are also of importance.

The Coulomb Equation

6.30 The angles of internal friction that have been observed for the clay soil tested range from 7.5 to 24 degrees. This spread encompasses the values normally associated with clay soils and have been brought about only by changes in the adsorbed cation and concentration

of salts in the pore water.

6.31 The relationship between shear strength and normal stress for a soil is given by Coulomb's equation:

$$S = C' + \sigma' \tan \phi'$$

$$= C' + (\sigma - U) \tan \phi'$$

where: S = Shearing strength

C' = Effective cohesion (intercept of the Mohr envelope and the shear axis).

σ' = Effective normal stress

σ = Total normal stress

U = Pressure in the pore water incapable of resisting shear (usually considered as 'hydrostatic').

ϕ' = Angle of internal friction (angle that Mohr envelope makes with horizontal).

In several instances this equation has not yielded soil strengths compatible with observed behaviour in the field. Normally, shear strengths determined from field observations are lower than those obtained by laboratory testing of samples of the soil. Several attempts have been made to have the Coulomb equation yield lower values of shear strength. One way was to assume that the cohesion tends to zero over long time intervals and therefore eliminate it from the equation. Another method

was to increase the pore pressure term (U) by adding to it the swelling pressure exhibited by the soil in the consolidation test. However, it is suggested by the results of this thesis program that there could be a considerable discrepancy between the soil parameters, c' and ϕ', as determined in laboratory testing of soil samples and those existing in the soil mass in the field unless particular care is taken to ensure that neither the pore water salt concentration or the adsorbed ions are changed. If either of these two factors do change then the terms representing cohesion, angle of internal friction and pore water pressure in Coulomb's equation would be incorrectly assessed.

6.32 If some of the terms in Coulomb's equation are incorrectly assessed, what can be done about it. It is the intention of the succeeding paragraphs to discuss this question.

6.33 Generally the pertinent geotechnical properties of a soil are a function of the surface activity of the clay-size particles. The surface activity will vary as the number of clay particles present and with their mineralogy. The hydrometer analysis provides a measure of the former characteristic. If a second hydrometer analysis is carried out on a sodium modification of the same soil then an indication is obtained of the increase in surface area available due largely to particle

cleavage. If there is little difference in the two hydrometer analyses one could assume that the exchange of one cation for another on the exchange complex would probably not have serious consequences. Compare, for example the results of the hydrometer analysis (Plate 1) and the Mohr envelopes (Plates 12 and 14) of the magnesium and calcium modifications with similar results for the natural soil.

6.34 The salt content of the pore water is also significant and should be determined. The test for soluble salts is simple consisting in thoroughly mixing about 200 grams of soil in 2 liters of distilled water and allowing the soil to settle out. An aliquot portion of the clear supernatant water is evapourated and the salt content calculated. An indication of the influence on strength of variations in pore water salts is given by Plate 15 and Appendix L.

6.35 Other tests, not normally associated with a soils laboratory, are cation exchange capacity, flame photometer and x-ray diffraction analyses. Undoubtedly the results of these tests are valuable but may be expensive. The cation exchange capacity is a more direct measure of the activity of the clay sizes. The flame photometer shows the proportions of each cation that is adsorbed on the clay complex and indirectly the salt content of the pore water hence one can assess the

probable behaviour of the soil. X-ray diffraction studies will yield approximate percentages of each of the main clay mineral classes present. The presence of members of the montmorillonite family are indicative of large changes in geotechnical properties as physical-chemical changes occur.

6.36 A test that appears to be of considerable value and that merits more discussion than is possible here, is the consolidation test including observations of swelling characteristics. The film of adsorbed water that surrounds the clay particles must be effective in preventing actual mineral to mineral contact. There can only be contact between the adsorbed water hulls. In Paragraph 6.14 it was postulated that swelling pressures arise from increasing thicknesses of the adsorbed water film. The thickness of the water hull is dependent on the salts of the pore water as well as the adsorbed cations. It, therefore, would appear that much valuable information concerning the properties of the soil could be obtained by carrying out consolidation and rebound tests on a soil with various cations and anions in the water available for rebound. A soil that reveals large variations in compressive or swelling index would also have a large range of strength parameters under conditions of changing adsorbed cations or pore water salt concentration.

6.37 Until further research provides a new or improved laboratory tests, it appears that procedures in common use in a soils laboratory need to be augmented by tests on modifications of the soil to gain a more complete knowledge of the soil behaviour. The modifications may comprise a change of the salt content or a partial saturation of the exchange complex with a single cation. Additional and valuable information can be obtained from x-ray diffraction, cation exchange and flame photometer analyses.

Complicating Factors

6.38 The application of the work of the soil physicist is assuming more and more importance in Soil Mechanics. Research appears to be gaining momentum and it is to be expected that there would be a wide divergence of opinion in the interpretation of results. It is the purpose of this and the following paragraphs to try to enumerate a few of the complications that exist when a soil-water system is considered.

6.39 Water in itself is a complex structure (40, 49) possessing unusual properties such as an abnormally high melting point, heat of fusion, boiling point, heat of vapourization and specific heat. The high values of these quantities are attributed to the extra energy required to break hydrogen bonds. The unusually high viscosity of water

has the same basis as does the elevated dielectric constant.

6.40 Low, in a recent paper (40), discusses the subject of clay-water interaction and presents the following ideas as a working hypothesis for this phenomenon (see Paragraph D14, Appendix D). There is an adsorbed layer of water on the surface of a clay particle having a definite structure. The strength of this structure depends on hydrogen bonds and a tendency toward covalent bonds when the adsorbed water is linked to an oxygen on the surface of the clay crystal. The rigidity of the adsorbed water structure decreases with increasing distance from the particle surface. The adsorbed cations tend to disrupt the structure of the adsorbed water to greater or lesser extent depending on the hydrated size of the cation. The degree of disruption may range from nearly complete to practically none.

6.41 Superimposed on the preceding complexities are other active forces in a clay-water system. There are Coulombic forces of attraction and repulsion between the charged bodies present. There are clay surfaces and edges, free and adsorbed cations and anions all possessing charges and presumably all interacting. Further, the charges on a clay particle are not evenly distributed over the surface but may have points of concentration. The cations and anions all differ in size one

from another as well as in their hydrated and unhydrated radii. Thermal energy and the infinite variety of interparticle relationships would appear to compound an already complex system. In most instances, all that can be experimentally measured is the net effect of these forces.

Summary

6.42 In summarizing this chapter it is only necessary to reiterate the decisive role played in the strength concepts of a cohesive soil by changes in the salt content or the cations adsorbed by the clay complex. The plot of the logarithm of compressive strength and the virgin compression branch of the pressure-void ratio curve (Plate 23) is an illustration of the influence of these factors. The swelling pressure was suggested as stemming from increases in the thickness of the water adsorbed on clay particles. The level of salts present in the pore water under field conditions is of importance in accounting for its behaviour. It was stated that a soil mass in the field should be considered as a dynamic equilibrium consisting of the soil and its environment. Several instances of salt concentration in the pore water were discussed. The adequacy of laboratory tests were considered briefly and it appears that in the case of some soils these tests must be augmented by others. Clay water systems are a complex interplay of many factors and probably all that can be measured is the net effect of these factors.

CHAPTER VII

CONCLUSIONS

7.1 The shear strength characteristics of a clay soil may be affected when a single cation species occupies a high proportion of the exchange complex or when the concentration of salts in the pore water changes. Due to the residual salts in the pore water of the specimens, the results of this investigation are more of a qualitative nature than quantitative when applied to clay soils generally. The following conclusions arise from the work of this program.

7.2 The angles of internal friction observed for the soil modifications tested are presented in Table 8. Considering the values of effective friction angles obtained from Plate 15 at zero salt content, it is noted that the potassium, calcium and magnesium modifications are fairly closely grouped about a value of 23 degrees, the natural soil is significantly less having a value of 14.5 degrees and the sodium soil is very low with a value of 7.5 degrees. It is suggested that the adsorbed water hulls are thin for high angles of internal friction and thick for low angles.

7.3 The strength characteristics of a clay soil may also be influenced by the presence of salts in the pore water. For the potassium modification there appears to be no changes in the angle of internal friction as the salt content changes. The friction angles of the calcium and magnesium soils appear to change to a small extent but the changes may be masked by the degree of accuracy of the results. The angle of internal friction for the natural soil increases as the salt content increases. It is suggested that the presence of both the calcium and magnesium cations on the clay complex are responsible for this behaviour. The increase in the friction angle for sodium as the salt content increases is very pronounced. The reason postulated for this is the thinning of the adsorbed water hulls due to the presence of salts in the pore water. A small increase in the salt content of the pore water results in an appreciable increase in the angle of internal friction.

7.4 The time rate of consolidation observed during the triaxial test varies over a wide range as the adsorbed cation and the pore water salt content change. Times to 100% theoretical consolidation are listed in Table 6. For the condition of no salt in the pore water, the time for sodium to consolidate is in the order of 400 times that for the calcium soil and 130 times that for the magnesium modification. Potassium soil is about 10 to 30 times longer and the natural soil 3 to 10 times

longer consolidating than the soils of the divalent cations. It is concluded that the time for consolidation to occur depends on the cation occupying the exchange complex and the concentration of the salts present in the pore water. For the condition of no salts in the pore water, adsorbed monovalent cations result in much longer consolidation times than do adsorbed divalent cations. As the concentration of salts increase the consolidation time decreases for the sodium, calcium and magnesium soils and increases for the potassium and calcium soils.

7.5 The adsorbed cation complex also exerts an influence on the observed specific gravity of the soil solids of the material used in this investigation. The presence of salts and the total amount of water adsorbed on the clay particles when the soil is in the air dry state are both shown to increase the observed value of the specific gravity. The sodium modification has a value of 2.79, slightly higher than natural value of 2.78. The lowest value is that for magnesium of 2.68. The other modifications lie between these extremes.

7.6 The cation adsorbed on the clay complex is indirectly responsible for variations in the per cent clay sizes determined in the hydrometer analyses of the soil modifications used in this work. The sodium ion has the highest percent clay sizes while calcium and magnesium

modifications are a little higher than the natural soil.

7.7 It is concluded that the straight line relationship for the plot of moisture content versus the logarithm of the compressive strength, Plate 19, exists only if the salt content of the pore water of the various samples is constant. Different salt contents yield different slopes to the curves on this plot, and, generally, the higher the salt content the flatter the slope. Further evidence of the salt content of the pore water is indicated by the plots of the logarithm of the compressive strength and the virgin compression branch of the pressure-void ratio curve versus the moisture content, Plate 23. It is tentatively concluded that parallelism only exists if the level of salts in the pore water are the same for each sample.

7.8 The explanations of the observed test results presented in Chapter V and the discussions of Chapter VI are primarily based on the thickness of the water film adsorbed on a clay particle. Thin water films are responsible for high friction angles, low specific gravity of soil solids, and flatter slopes of the curves on the moisture content-logarithm of compressive strength plots. As the adsorbed films increase in thickness the angle of internal friction decreases, the specific gravity of the soil solids increases and the slope of the moisture

content-log compressive strength curve becomes steeper. The range of values of the angle of internal friction for the soil tested in this program have bracketed the tranditional values assigned to 'cohesive' soils. It would appear, therefore, that the major conclusion of this report is that many of the geotechnical properties of a cohesive soil are strongly influenced by the salt content of the pore water and the nature of the cations adsorbed on the clay complex.

CHAPTER VIII

RECOMMENDATIONS

8.1 Each of the following paragraphs outlines a single, general recommendation that has arisen from the work of this program.

8.2 It has been shown that the salt content in the pore water influences the strength characteristics of a clay soil. In order to determine the relationship existing between the shear strength and the concentration of the salt solution it is recommended that the following investigation be considered:

(a) a series of tests be conducted on each soil modification whose only variable is the concentration of the salt in the pore water.

(b) a series of tests be conducted to ascertain the effects of the anion in the pore solution.

The investigations could take the form of consolidated-undrained triaxial tests with pore pressure measurements conducted on a homionic soil with controlled amount of salts in the pore water. The salts being made up of the appropriate cation.

8.3 It has been suggested that the gradual replacement of one cation by another may have no influence on the adsorbed water film development up to a certain point at which there is a sudden abrupt change in the stable thickness of the oriented water. It has also been suggested that magnesium may enhance the effect of the sodium ion when both are adsorbed by a clay soil. In addition to practical considerations, both of these points are of interest because the soil modifications of this program were not truly homionic. Therefore, it is recommended that:

(a) a series of triaxial tests be carried out on a calcium soil with varying amounts of sodium replacing the calcium on the exchange complex. The pore water should be free of salts to eliminate this variable.

(b) a similar series of tests be conducted on a magnesium soil.

(c) a series of tests similar to (a) above be conducted but with magnesium replacing the calcium.

8.4 Since cohesion may be an integral part of the strength of a soil, this property should be investigated. In addition to any data observed during the programs suggested in previous paragraphs it is recommended that a separate program be carried out on the soil modifications using the technique of Schmertmann and Osterberg (56) to study the variation of cohesion and angle of internal friction with strain. These

results may also be coupled with unconfined compressive strength tests.

8.5 The chemicals used in this program are largely inorganic but there are many organic compounds that influence the behaviour of the clay colloid. It is recommended that shear strength variations with a few typical organic compounds be investigated. Specifically, a series of tests may be conducted on a soil leached with water from a muskeg (See Paragraph 6.15).

8.6 The swelling possibilities of the soil modifications used in this program were referred to briefly in Paragraphs 6.14 and 6.36. It would appear to be both useful and feasible to investigate the consolidation and swelling characteristics of clay soils in the presence of various salts. It is recommended that a series of consolidation tests be conducted on homionic soils with various concentrations of salts available during rebound. Each test may consist of several load-rebound cycles, each successive cycle having a more concentrated solution surrounding the specimen. A subsequent series of tests may vary the cation in the solution. Such a test series may also be carried out on 'undisturbed' samples.

8.7 Although not arising solely from this program, it is apparent

that some knowledge of the fundamental concepts of clay mineralogy and soil physics is necessary if one is to understand more fully the geotechnical behaviour of fine grained soils. It is, therefore, recommended that consideration be given to inclusion of the fundamentals of clay mineralogy and soil physics in the training of engineers specializing in the field of soil mechanics.

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APPENDIX A

PROGRESS REPORTS ON THE EFFECTS OF CATIONS ON THE SHEAR STRENGTH OF AN ALBERTA CLAY SHALE

APPENDIX A

FIRST PROGRESS REPORT ON EFFECTS OF CATIONS ON THE SHEAR STRENGTH OF AN ALBERTA CLAY SHALE

The Problem Being Studied

A1.1 Simply expressed the problem is - what effect does a change in cations have on the strength of clay shales. It has been shown by the work of others (63)* that the strength of a remoulded soil is affected by the ions adsorbed on the clay particles.

A1.2 There are two major considerations behind this work. One of these is to determine whether or not a strength loss in field problems, particularly landslides, is attributable, at least in part, to changes in the adsorbed ion complex. Stress-strain characteristics form a part of this phase. The other consideration is whether or not advantage can be taken of any possible improvements in the shear strength that may be forthcoming in this work.

A1.3 It is appreciated that the early phases of this work are exploratory in nature and will lead to a specific task or problem suitable for more detailed examination.

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* Numbers in parentheses refer to List of References in the Bibliography of this thesis.

Soil Being Used

A1.4 For the very early phases and before the samples from Test Hole A were available, 5 shelby tubes from Test Hole 140 at Dunvegan were obtained. These samples were obtained from depths of 75, 85, 95, 105 and 115 feet. The engineering properties of this material are similar to those of the soil from Test Hole A and are reported as being (62):

Liquid Limit	83.2%
Plastic Limit	29.9
Plasticity Index	53.3
Shrinkage Limit	18.4
Finer than 2 microns	73

A1.5 The samples used in later work were extracted from boreholes advanced by continuous sampling with thin walled shelby tubes. Test Hole A was 91 ft. deep and Test Hole B was 25 ft. deep. The hole logs were determined from the samples as they were extracted from the shelby tubes. A moisture content sample was obtained from the centre of each specimen. The moisture content profile for both holes is plotted on Plate A1. Shear planes, or slickensides, were observed at depths of 20 to 25 feet and 62 to 64 feet below ground surface. The entire series of samples is interspersed with silt seams and partings which are generally about 4^{o} to 6^{o} off horizontal. The specimens split extremely easily along these silt seams. At a depth of about 60 feet the silt lenses were 2 to 4'' thick. The remainder of the soil was a dark grey clay of the following average characteristics (62):

Liquid Limit	81.7%
Plastic Limit	30.4
Plasticity Index	51.3
Shrinkage Limit	16.6
Finer than 2 microns	72

A1.6 The borehole is located to the south and east of the recent slide area at Dunvegan, Alberta. Dunvegan Creek occupies and is re-excavating a preglacial valley that became infilled during glacial and interglacial periods by fluvial and lacustrine deposits. The erosion by the present Dunvegan Creek has exposed the original preglacial wall on the west side but the east wall is still made up of the interglacial and glacial fill. This latter material is apparently unstable as is indicated by the recent slide and the clearly visible scars of previous slides. The soil being used in this investigation is comprised of the infilled material.

A1.7 That portion of the soil less than two microns in diameter was shown to have the following approximate mineral composition (62):

Mineral	Test Hole A	Test Hole 140
Montmorillonite	10 to 20%	25 to 35%
Illite	30 to 45	25 to 35
Chlorite	30 to 45	0
Kaolinite	5 to 15	25 to 35

A1.8 The results of ion exchange determinations revealed the following average distribution of adsorbed ions:

Test Hole	Percent of Exchangeable Positions Occupied per 100 Gm. Soil				Exchange Capacity m.e. per 100 gm.
	Na	Ca	K	Mg	
140	2.2	67.3	5.1	25.4	27.1
A	1.9	67.2	5.3	25.6	26.3

Investigational Procedures

A1.9 The first procedure was generated by a line of reasoning similar to the following. In nature, the clay shales in many instances are in a state of rebound due to release of load from retreat of the glacier and from subsequent erosion of the land. The latter effect is particularly evident in valleys. While these soils are rebounding they must be taking up water and the water should normally contain some cations in solution. There would, therefore, be a tendency for ion adsorption or exchange to take place. It was considered that the simplest way to simulate this natural condition was to consolidate the specimens in a triaxial cell at a fairly high all round pressure; then to allow rebound at a lower pressure. During the rebound phase a solution containing a given cation was made available to the soil. In the early tests the specimens were consolidated at a pressure of 8kg/cm^2 and rebound was allowed to take place under pressures of 2, 4 and 6 kg/cm^2. A 1 Normal solution of the chloride of the salt of the desired cation was allowed to be

taken up by the soil. The progress of consolidation and rebound were followed by plotting the readings of a burette, which was connected to the specimen, against the logarithm of time. Completion of the consolidation and rebound phases were determined from these plots. After completion of the rebound the specimens were failed by a consolidated quick test with pore pressure measurements. Although the results indicated some trends, they were not sufficiently clear to be conclusive. For clarification the second procedure was begun. The results will be discussed later in this report.

A1.10 The second procedure was based on the following considerations. The average amount of solution being taken up by the soil was 1.35 cc. of solution with a range from a minimum of 0.31 cc. to a maximum of 2.75 cc. Since a 1 Normal solution was used, these figures also represent the milliequivalents (m.e.) of cations entering the soil sample. A soil sample weighs about 150 grams and contains about 0.75 m.e. of exchangeable adsorbed Na ions. On the average, therefore, and assuming all the ions entering the sample are adsorbed, the Na ion increases to about 1.35 + .75 + 2.1 m.e. Percentagewise, this is an increase in Na ion from 1.9% (Paragraph A1.8) to 5.3%; in the Ca ion from 67.2% to 70.6%; in the K ion from 5.3% to 8.3% and in the Mg ion from 25.6% to 28.9%. It would appear that these increases may have some effects

but natural variations would tend to mask them. There was also the question as to what percentage of the ions available in the rebound phase were in fact being adsorbed. Ion determinations on the flame photometer were inconclusive on this point but flame results indicated that the foregoing calculations give predictions that are a little high. Further, flame photometer results indicated that the ion available to the soil during rebound was not being uniformly adsorbed throughout the specimen but was in greater concentration at the bottom than at the top of the sample. See for example, the last two lines of Table A4. (The Geonor cell does not have top drainage.)

A1.11 To achieve cation exchange it is necessary to remove from the soil specimen those ions replaced or an equilibrium may be set up as indicated by the Law of Mass Action. It was, therefore, decided to leach the specimens with a chloride solution of the desired ion. After a period of leaching, the specimens were tested in a consolidated-undrained triaxial test with pore pressure measurements.

Equipment and Techniques

A

A1.12 The soil samples were extruded from the shelby tubes immediately after their arrival at the University. After extrusion they were visually described for the hole log, a moisture content strip pared off, measured for length, then wrapped in silver foil and coated with wax.

Specimens from 68, 71, 72 and 73 feet were prepared for testing very shortly after extrusion from the shelby tubes by trimming them down to 36 mm. diameter by 90 mm. (approximately) long on the vertical soil lathe. The trimmings from these samples were used by P.A. Thomson in his work. Some specimens could be trimmed by rotation and a chisel but others split before trimming was completed. These were put back together and trimming completed by vertical strokes of a wire saw. On completion of the trimming, the samples were wrapped in silver foil and stored in the moist room.

A1.13 In the tests the samples were trimmed to 80 mm. long in a cradle, weighed, carefully measured and placed in the triaxial cell. Later specimens had 5 wool wicks inserted symmetrically to act as internal drainage. A hole was punched first with a 1/16" diameter piano wire mounted in a drill press. The wire was not rotated but the vertical action of the drill press allowed the punching of a hole in a continuous quick thrust. These holes were only 70 mm. long, that is they reached to within 10 mm. of the top of the specimen. Holes going entirely through the specimen did not seem to be as effective in pore pressure measurement. On occasion, hair cracks were observed along the sides of a specimen adjacent to one of the holes. After the piano wire had been withdrawn a single thickness of ordinary knitting

wool was stuffed into the hole by means of a darning needle which had the eye broken thus forming a notch to catch the wool. Care must be exercised to place the end of the sample from which the wool protrudes against the porous stone.

A1.14 In the second phase of the work, the leaching of the specimens was carried out in two ways. First by trimming the specimen to fit snugly inside a 1 1/2" I.D. lucite tube, capping the tube and forcing the solution containing the ion through the sample under a pressure of 2 kg/cm^2. (See Plate A2(a)). This pressure was maintained for 1 to 2 weeks. The second method employed a triaxial cell that had top drainage. The sample was set up in the triaxial cell complete with wool wicks, filter paper and membrane, but the filter paper was kept about 10 mm. from the bottom of the specimen and folded across the top of the sample to make contact with the top drainage plate. The top drainage line was left open to the atmosphere. The bottom sample drainage line was hooked to a bottle containing 1 N solution of the chloride of the ion desired. The cell line was hooked to a bottle of distilled water. Both these bottles were placed at an elevation of about 12 feet above the cell. This was left for about 2 weeks (Plate A2 (b)). After leaching, the samples were removed, stripped and checked for measurements. If necessary, the specimen was trimmed down before being placed in the

Geonor equipment.

A1.15 After weighing, the specimens were wrapped in a saturated, slotted filter paper which lapped over the saturated porous stone. This was mounted on the pedestal of the triaxial cell and a single standard prophylactic membrane rolled upwards nearly to the top. The top cap was centred and the membrane rolled over it. A layer of silicone grease was applied to the membrane and a second one placed over this. Seals were completed by two "O" rings on the base pedestal and two on the top cap. During this procedure obvious trapped air bubbles were worked out gently with the fingers.

A1.16 The triaxial equipment used is Geonor supplied from Norway. Its operation is described in detail elsewhere (46). The following techniques were used during testing. After the specimen had been sealed in the cell and the cell filled with water, it was isolated by closing the appropriate valve. The constant pressure cell was prepared for the desired pressure and it was isolated. A burette was connected to one of the leads from the bottom of the specimen and the system filled with de-aired distilled water making sure all air was out of the lines. The valve to the burette was closed and the other line plugged. The cell was then brought up to pressure and the constant pressure cell valve opened. The time and reading on the burette were noted and the

valve opened hence starting consolidation. Readings were taken at doubled time intervals starting at 0.25 minutes and a plot was kept as consolidation proceeded. Invariably air would show up in the line from the sample to the burette but took 15 to 25 minutes to enter the burette. At this point the pore pressure valve was rigged quickly to the other drainage line from the specimen and using the pore pressure set-up water was sucked from the burette back through the line and the air was removed from the system. This operation took 1 to 3 minutes and as soon as it was completed the water level in the burette was adjusted by extrapolation of the consolidation curve. Primary consolidation was completed in 100 to 120 minutes in those samples having wool wicks.

A1.17 Normally, the samples were set up one day and tested the next. For testing the burette was removed and the pore pressure apparatus rigged to the drainage line. A back pressure of 2 kg/cm^2 was applied to the cell and to the pore water of the specimen simultaneously and in small increments over a period of about 5 to 10 minutes. The back pressure to the pore water was rigged through a mercury manometer and to a constant pressure cell, i.e. the mercury manometer read zero with 2 kg. back pressure on each leg. Pore pressures therfore, could be read with an accuracy of 0.01 kg/cm^2. The test is of the constant strain type and rates of loading of 1.25% and 2.08% axial strain per

hour were used. The latter rate results in a test of 5 to 6 hours duration. Readings of time, pore pressure and vertical load were taken at strain increments of 0.5% or less and plots of axial strain versus deviator stress, effective stresses, pore pressure and the principle effective stress ratio were carried along with the test. Progress of loading could thus be followed and mistakes or erratic readings could be checked quickly. The maximum ratio of principle effective stresses was chosen as the criterion of failure and all stresses reported were taken from the curves at the strain at which the maximum ratio occurred. The tests were carried on well past the point of maximum stress ratio, normally until a distinct shear plane was visible on the specimen.

A1.18 On completion of load testing, the specimen was removed from the triaxial cell, stripped of membranes and filter paper and a sketch of the failure made. The specimen was weighed and its volume determined by mercury immersion. A slice about 1/2" thick was taken off each end of the specimen and air dried for ion determinations. The remaining centre portion was used for a moisture content determination. Calculations for initial and final void ratio, moisture content and degree of saturation were made.

A1.19 Ion exchange determinations were carried out on the soil samples in the laboratories of the Alberta Research Council in the Biological

Sciences Building. Dr. A. Mathieu was extremely helpful in making the equipment available and in explaining, in detail, the techniques. Briefly, the procedure is as follows: a 3 to 5 gram sample of pulverized air dried soil is weighed out and placed in a test tube with 40 ml of 1N ammonium acetate (NH_4Ac). After a shaking period the slurry is transferred to buckner funnel and aspiration applied. The sample is leached with successive portions of NH_4Ac and the leachate transferred to separate bottles and kept. The sample is washed with ethyl alcohol, then leached with 1N sodium chloride, ($NaCl$). The $NaCl$ leachate is distilled into a boracic acid solution and the ammonia titrated with sulphuric acid. The processes that occur are removal of all ions adsorbed (and in the pore water) by ammonia (NH_3). These are retained in the first leachate. Washing with alcohol removes any NH_3 ions from the pore spaces of the soil. Leaching with $NaCl$ removes the NH_3 ions previously adsorbed and transfers them to the second leachate. Distillation and titration for NH_3 gives the exchange capacity of the soil. This is reported in milliequivalents (m.e.) per 100 gm. of soil.

A1.20 The first leachate, i.e. that containing the original adsorbed ions was subjected to flame photometric determinations to determine quantitatively the amounts of K, Ca, Mg and Na cations present. Generally, this procedure is based on the fact that a given ion imparts a

certain unique colour to an acetylene-oxygen flame. The characteristic wave length emitted by the flame under the influence of the ion is isolated by machine settings and the intensity of the emission measured. This is compared to a standard curve and quantitative values for the amounts of ion present are calculated. These values are also reported in m.e. per 100 gms. of soil.

A1.21 It would appear that the sum of the flame photometer results (i.e. the total m.e. of Na, K, Ca and Mg ions) should equal the exchange capacity of the soil. However, this is not the case and usually, the flame results are greater than the exchange capacity. The reason for this is the presence of cations in the pore fluid. These can be determined if enough soil is available but unfortunately this is not always the case. However, the dissolved cations may be considered as not too variable over depth range of several feet, hence one determination of the cations in the pore water can be used to 'correct' the flame determinations. It will be noticed that the hydrogen cation has not been mentioned. This is due to the fact that the natural pH of the soil is in excess of 7.5. Therefore the H ion is considered to be absent.

A1.22 The X-ray diffraction and interpretation of results were carried out by the staff of the Alberta Research Council on glass slides prepared from the minus 2 micron fraction by the author and P. Thomson.

The interpretation was discussed in some detail with A.R.C. personnel. The procedure is based largely on the fact that reinforcement of incident X-rays by reflected rays from a crystal are a function of the incident angle and the 'd' spacing (or distance between layers of atoms) of a crystal. Each mineral has a characteristic angle of incidence at which reflection is a maximum. For some minerals (e.g. Illite and Montmorillonite) the angles of maximum reflection are very close together, hence difficult to distinguish. The minerals are modified by heat treatment and by glycolization. The former destroys the Kaolinite lattice and the latter causes montmorillonite to swell. Illite is not affected by either treatment - hence separation of these minerals is facilitated. The intensity of the reflection is a rough measure of the amount of the mineral present.

A1.23 During the programme, a series of two tests were carried out using a procedure that maintained a constant vertical effective stress (56). This type of test was designed to investigate the variation of the angle of internal friction and of cohesion as the strain increases. The vertical effective stress is controlled by varying the pore pressure. Normally two effective stresses are chosen such that the difference between them is about 25% of the overburden pressure and the upper value is somewhat less than the predicted maximum. The test is started and allowed to

continue as a normal test until the lower of the chosen effective stresses is reached. This stress is maintained by increasing the pore pressure. Several readings are taken until a portion of the deviator stress versus strain curve is established. A sudden lowering of the pore pressure (by the difference between the chosen effective stresses) will increase the effective stress. The pore pressure is juggled until such time as the desired effective stress occurs. This upper stress is maintained until a portion of the deviator stress versus strain is established. Increasing the pore pressure causes the stress to return to the lower curve and the cycle starts again. This is continued to failure of the specimen. Full curves are now drawn through the portions established. With this data 2 Mohr circles can be constructed for any given strain. From the envelope to these circles, the angle of internal friction and the cohesion can be determined for the chosen strain.

Results and Conclusions

A1.24 The data from the first series of tests (the rebound procedure) are summarized in Tables A1 and A2 and Mohr circles and Principle Stress Ratios on Plates A3 and A4. The results of the second series are in Tables A3 and A4 and on Plates A5 and A6. The constant principle stress test results are plotted on Plates A7 and A8.

A1.25 Considering the results of the first series of tests, a study of

Tables A1 and A2 indicates the following trends when the tests are considered in groups based on consolidation pressure. In each group the natural soil sample had the lowest pore pressure and the highest minor principle effective stress. The major principle effective stress was highest in 2 of the 3 groups for the natural soil. In 2 of the 3 groups the sample having highest principle effective stress had the highest amount of adsorbed calcium ions and the lowest amount of adsorbed sodium ions. The reverse also appears to hold, that is the lowest effective stresses are associated with the highest amounts of adsorbed sodium ions and the least amount of adsorbed calcium. Tentatively then, it would appear that the presence of NaCl salts in the pore water as well as adsorbed ions increases the pore pressure, that effective stresses can be increased by increasing the amount of adsorbed calcium or decreasing the adsorbed sodium and that the effective stress can be reduced by decreasing the calcium adsorbed and increasing the sodium adsorbed. The initial and final moisture contents, void ratios and degrees of saturation cannot be compared due to these data being unreliable. The lowest angles of internal friction also appear to be associated with the highest amounts of adsorbed sodium. The two Mohr envelopes plotted on Plate A3 also support the observation.

A1.26 Reviewing Plate A4, Principle Stress versus Compressive

Strain, the first point noticed is the curves of samples 95-1 and 115-1 being much higher than the others. Just why this should be is not known and since thereis only one sample of each any postulates cannot be checked. One explanation is the existence of a weak zone which is represented by these samples. The three curves from the rebound-with-water samples (75-1,2,3)(dashed lines) are grouped and are above those for the 3 samples from 105' rebounded with NaCl. This relationship confirms that on Plate A3; that is a lower angle of internal friction for rebound with NaCl. Another point is the fact that the ratio of principle stresses is generally higher at strains less than 1% for sodium rebounded than for water rebounded. This means that the strength is mobilized more quickly at low strain for sodium rebounded soil. This is confirmed by comparing the principle stress versus strain curves.

A1.27 The second series of tests (Tables A3 and A4) also shows up some interesting trends. The most obvious of which is the marked increase in the amount of adsorbed ion with which the sample was leached. The first point is the Mohr envelope from the 75' samples. The angle of internal friction is 17° which is higher than the "sodium" envelope from T.H. 140 at 105°. This tends to confirm the effect of sodium ions on strength. Table A5, which is a rearrangement of the data from 73 and 74 feet of Table A4, illustrates the ion effects more

clearly. The table is arranged to have the most desirable soil properties on the left and the least desirable on the right. The first five entries are considered to be the most important. It will be noticed that the most desirable soils are the calcium modification and the natural soil. These samples have the most adsorbed clacium ion with the former soil being nearly homionic. These results tend to confirm those of the first series of tests. However, the least desirable properties are those of the potassium sample with the sodium sample occupying the third place. The relative positions of calcium and sodium are thus confirmed. Since there is only one test, the position of potassium is open to question particularly in view of the relationship given by Tschebotarioff (63).

A1.28 The results of consolidated-undrained tests from T.H.'A'. are plotted on Plate A5 and the Ratio of Principle Effective Stresses versus Strain on Plate A6. There appears to be little that can be gleaned from these graphs that has not already been covered. The top 3 curves of Plate A6 are from the same shelby tube and it is interesting to note the long down curving of the potassium sample when compared to the others.

A1.29 Plates A7 and A8 present the results of the constant principle effective stress test. It is presented for interest and shows, in a general way, how the cohesion and angle of internal friction vary with strain.

The results are similar to those obtained by Schmertmann and Osterberg (56). The author learned the technique and also that the proving rings of the Geonor are not entirely suitable for this special test.

A1.30 The general conclusions that are indicated by the work thus far are that the soil strength can be increased by increasing the amount of adsorbed calcium and decreasing the adsorbed sodium. The reverse also appears to hold.

General Discussion

A1.31 Several difficulties or troubles were encountered during the testing program. The most noticeable of these is membrane leakage during consolidation. Leakage is shown up by the consolidation curve (burette reading versus log of time) when the curve does not flatten out but continues downward often at a sharper degree of curvature. As experience with the equipment was gained the incidence of membrane leakage decreased but still happened occasionally. In these cases the cell was stripped down and another membrane added or the membranes replaced. The sample was then reconsolidated and the test carried on. If reconsolidation did not occur, membrane replacement was repeated. Leakage did not occur in any one cell more often than in any other thus the fault was with the membrane or the technique. There are two major troubles arising from membrane leakage - one being the possible ingress

of water to the specimen and the other the difficulty of arriving at the Geonor Correction Factor. It was assumed that the moisture content at the end of reconsolidation was the same as that which would have occurred had there been no leakage. The Geonor correction factor is used in the calculation of the deviator stress. The original cross-sectional area, which is measured prior to consolidation, will be decreased by consolidation. If the volume change during consolidation is known and and isotropic, volume decrease is assumed, the cross-sectional area can be calculated. The volume change is measured by the change in burette readings. Thus, if leakage occurs it is impossible to tell how much water was squeezed out of the sample during consolidation. In these cases an estimate was made based on the anticipated time of 100% consolidation from other like samples. On occasion the sharp increase in curvature of the consolidation curve is indicative of when leakage became predominant. The tendency when leakage occurs is to increase the correction factor which in turn increases the strength. Just how much this increase is, is difficult to assess but it is believed to be small - probably in the range of 0.1 to 0.3 kg/cm^2 being the lesser value for low deviator stresses and the higher for high deviator stresses.

A1.32 One of the most inexplicable leakages occurred during the rebound phase of early tests. No leakage occurred at cell pressure of

8 kg/cm^2 but on rebound at 2 kg/cm^2 some specimens developed a leak toward the end of rebound. These were stripped down, membranes replaced and reconsolidated at the rebound pressure. The effects are the same as those discussed in the previous paragraph.

A1.33 It would appear that the volume changes as indicated by the burette readings should be the same as that calculated from the initial and final volumes. However, agreement between these two was approximately reached in only 2 of the last 8 consolidated-quick tests. In the remaining tests the discrepancy was 2 to 4.5 cc. In three of these tests the volume changes showed a decrease by burette readings and an increase by initial and final volume comparison. Just why this anomoly should exist is not known but possible causes could be back pressure application not being equal, a pore pressure existing at dismantling at end of test, clearing the lines of air at various stages of the test and during dismantling. Dealing with these four causes in turn, an unequal application of back pressure, i.e. cell pressure increase not equal to back pore pressure increase, would change the effective stress. If the back pressure applied to the sample was less, expansion would occur. The bourdon gauges have been checked against each other and the discrepancies that occurred are in the order of 0.05 kg/cm^2. At the end of the compression test pore pressures ranging from 0.21 to 1.35 kg/cm^2

existed. However, the two specimens that gave volume change agreement had pore pressures at end of test of very close to 1kg/cm^2, and specimens having a low residual pore pressure had just as big a discrepancy as those with high pore pressures. During consolidation air is cleared from the lines with the pore pressure apparatus. The pressure used to do this is only applied for a maximum of two or three minutes. It would appear that the effects of the decrease in effective pressure would be nullified during the remainder of the consolidation phase and the volume change that would occur during these 2 or 3 minutes must be small (estimated at 0.1 cc). The same arguments would apply to de-airing the line at the start of the compression phase of the test. When the specimen is dismantled it quickly goes from a test pressure (e.g. 4 kg/cm^2) to atmospheric. The final volume would thus tend to increase - hence the calculated volume change would thus be higher than it should be. However, there is no consistency between the volume changes as indicated by the burette and by calculation. In one case the former is high and in another case the latter is high. The weight changes occurring during the test tend to show closer agreement with the volume change as determined by calculation but again not with complete consistency. During two tests actual volume change occurring during the compression part of the test was measured and found to be 0.07 cc. From this discussion it appears that agreement of volume

change by the two methods is largely achieved by careful technique and, perhaps, a certain amount of luck. There does not seem to be any apparent reason for the discrepancy at this time.

A1.34 The interpretation of the test results is made somewhat questionable by the natural factors inherent in any undisturbed material. There will be a natural variation of void ratio, moisture content and strength both within a shelby tube and from shelby tube to shelby tube. This makes direct comparison difficult and quantitative analysis nearly impossible since the effect of any change in adsorbed ions cannot be separated from natural variations. Further these natural variations may reinforce ion adsorption effects or may counteract them. The determination of the ion exchange capacity is also subject to natural variation and duplicate determinations that are within 5% of each other are considered good agreement. The results in this report are averages of duplicate samples and in some cases triplicates. The question that arises, however, lies in the fact that in some instances the difference in the amount of a given adsorbed ion in one sample is not much greater than in another sample. The significance of this slight difference is open to question. To get around these points a large number of samples must be compared, that is the problem must be approached somewhat statistically. It is considered that far too few tests have been conducted

in this investigation to date.

A1.35 There are a few other points that should be mentioned. A study of Table A3 reveals some anomalies, for example the final moisture content being greater than the initial. In some instances (e.g. at low cell pressure) this is not out of line since swelling did occur during testing. However, in other instances the burette indicates volume decrease and measurement volume increase. It is felt that most of these anomalies are due to early faulty technique. Another point is the correction that can be applied to the quantitative ion determinations for the ions in the pore water. This is referred to as the Soluble Salts determination. The test consists of bringing the soil up to a little above the liquid limit by adding distilled water, then extracting some of the water by aspiration. This water is then examined by the flame photometer for the amounts of the various ions. These amounts are then subtracted from the flame determinations of the leachate from the ion exchange determination (see Paragraphs A1.19 and A1.20). Several soluble salt determinations were carried out but the amount of soil available was only one-fifth to one-quarter of that recommended. This casts doubt on the results. Further, in a few instances the amount of a given ion in the pore water was in excess of that adsorbed. If this correction were applied a negative answer would result which is impossible. The

situation could possibly be interpreted as meaning zero amount of the particular ion adsorbed but this seemed unlikely in view of other determinations.

A1.36 Another point worthy of mention is the disturbance to the sample while it is undergoing leaching. Water is made available and thus some rebound must be occurring and the water content must be increasing. This cannot be avoided with the present procedure.

A1.37 Homionic conditions appear to be difficult to achieve in an undisturbed sample though this is by no means certain. In the samples tried the sodium ion was increased to 80.5%, potassium to 56% and the calcium to 98% (see Table A4). However, these are not corrected for soluble salts, hence do not reflect actual amounts of these ions adsorbed on the clay particles. It is assumed that the amount adsorbed is of the same order of magnitude. The 98% in the case of calcium was achieved by a different leaching procedure and had this procedure been used for potassium and sodium the amounts adsorbed may have been higher. Even if homionic conditions were achieved, the natural variations would still make interpretation difficult. Also the effects of one ion upon another could not be properly investigated. The immediate conclusion arising from this is that remoulded specimens be prepared from an homionic soil.

A1.38 Referring to Paragraph A1.25 it may be instructive to consider the increase in pore pressure during the compression phase of testing. Pore pressure may be defined as the difference between the total pressure and the effective (intergranular) pressure. Hence it follows that as the pore pressure increases the effective stress must decrease and, of course, the reverse relationship holds. Since intergranular pressure may be considered as a grain to grain contact, and increase or decrease in effective stresses can be visualized as an increase or decrease in grain to grain contact. All soil particles in a natural state have an adsorbed water film surrounding them. The thickness of these water films is influenced by the adsorbed ion, particularly the hydrated radius of the adsorbed ion. Tschebotarioff (63) states that the number of water molecules adsorbed by a cation increases with increase of the ionic charge and is a function of the ionic radius. The hydrated radius of the calcium ion will be slightly larger than that of sodium but two sodium ions can be adsorbed for each calcium cation. It follows, therefore, that sodium cations will result in a particularly thick film of adsorbed water around the soil particle. It is felt that this water is in a solid state (though still mobile!). The implications of this, as far as pore pressures are concerned, may be as follows. As sodium is adsorbed in increasing amounts, the water film around the particle must be increasing in thickness. Particles, therefore, must be being forced

apart hence less intergranular pressure. Also, as the water film grows the volume available for free water decreases hence pore pressure increases. These are the same thing and are reflected in pore pressure measurements so far in this work.

Future Operations

A1.39 The influence of natural factors, both in undisturbed samples and ion determinations will be resolved by comparison to remoulded samples of an homionic nature. This is particularly the case when dealing with the insensitive soils that have been used so far in this work. When an homionic soil has been tested it is possible to study the influence of various other ions by altering the homionic state to a state in which two types of cation are adsorbed. With this idea in mind a testing program is being set up.

A1.40 It is proposed to create the homionic state by leaching pulverized soil with an aqueous solution of the chloride of the desired cation. The first attempt will be to have the soil in a three inch diameter lucite tube with a filter at each end (porous bronze plate or ceramic discs). The solution will rise upward through the tube, just fast enough to prevent clogging of the upper filter. The amounts of soil and strength of salt solution will have to be decided somewhat experimentally. Inlet and overflows will also have to be worked out but are not considered

serious details.

A1.41 When exchange has taken place, the soil water solution can be evaporated to a consistency near the liquid limit. Specimens for triaxial testing will be prepared from this slurry in 2" diameter lucite tubes by consolidation under a load of 1 or 2 kg/cm^2. It is anticipated, however, that some experimentation will have to be carried out to iron out procedures.

A1.42 The triaxial samples will be tested in consolidated-undrained triaxial tests with pore pressure measurements. The Mohr envelope or Henckel plot will give the parameters for study.

TABLE A1

RESUME OF DATA FROM TEST HOLE 140

Sample	$\sigma_1 - \sigma_3$	$\bar{\sigma}_1$	$\bar{\sigma}_3$	P_p	σ_3	$\bar{\sigma}_1 / \bar{\sigma}_3$	μ %	Initial MC%	Initial e	Initial S %	Final MC%	Final e	Final S %	$\phi°$	Remarks
140-75'-1	2.88	4.98	2.10	0.10	2.2	2.37	6.75	38.1	-	-	35.2	.89	108	23.7	Rebound with H_2O
140-95'1	2.99	4.62	1.63	0.37	2	2.84	4.06	32.2	.94	82.3	30.0	.90	97.4	28.2	" " $KClO_3$
140-105'-1	2.20	3.83	1.63	0.37	2	2.35	3.81	29.5	.85	94.5	28.6	.79	98.6	24.5	" " NaCl
140-115'-1	3.46	5.03	1.57	0.43	2	3.20	5.54	30.0	1.04	79.6	31.9	.87	100	31.6	" " NaCl
140-75'-2	4.77	8.37	3.60	0.40	4	2.32	5.93	30.2	-	-	30.8	.89	94.4	23.0	Rebound with H_2O
140-85'-2	5.55	9.01	3.46	0.54	4	2.60	6.20	28.2	.85	90.5	28.6	.78	100	26.4	" " NaCl
140-105-2	3.45	6.76	3.31	0.69	4	2.04	4.76	29.1	.83	95.2	27.0	.74	99.1	19.8	" " NaCl
140-75-3	6.19	11.20	5.01	0.99	6	2.23	7.13	30.9	-	-	30.3	.31	103	22.4	Rebound with H_2O
140-85-1	4.70	9.09	4.39	1.61	6	2.07	4.80	33.7	-	-	31.5	.89	97.5	20.8	" " NaCl
140-105-3	3.82	8.62	4.80	1.20	6	1.30	8.37	29.0	.84	94.2	27.0	.75	98.0	16.5	" " NaCl

NOTES:

1. Data arranged on basis of consolidation pressure for direct comparison.
2. ϕ value based on zero intercept on shear axis.
3. For all specimens other than those at 105', calculations for e and S are based on an assumed original volume of 80 cc. and volume changes by burette summation. Figures for e and S are therefore not reliable.

(4) For specimens at 105' volume at end of test was determined by mercury immersion and original volume by calculation from accurate measurements.

5. All pressures in kg/cm²

TABLE A2

Results of Ion Determinations for T.H. 140

Sample	Ion % of TOTAL FLAME				Exch. Cap. M.E. per 100 gm.	Remarks
	Na	Ca	Mg	K		
105'	2.9	73.3	20.0	3.8	28.2	Natural Soil
75-2	2.4	83.6	12.3	1.7	20.4	" "
75' to 115'	3.0	80.8	13.7	2.5	23.0	Mix of Natural Soils
115	2.4	80.8	14.1	2.7	26.9	Natural Soil
105'-2	4.4	83.0	10.0	2.6	23.3	NaCl on Rebound
105'-3	6.3	75.8	13.9	4.0		" " "
95-1	2.8	81.2	11.0	5.0	21.6	$KClO_3$ on Rebound
85-2	3.2	78.7	15.6	2.5	24.3	NaCl on Rebound
115'-1	3.4	83.4	10.6	2.6	24.4	NaCl on Rebound

NOTE: Determinations not corrected for cations held in pore water.

TABLE A3

Resume of Data from Test Hole 'A'

								INITIAL			FINAL				
Sample	$\sigma_1 - \sigma_3$	$\bar{\sigma}_1$	$\bar{\sigma}_3$	ρ_p	σ_3	$\bar{\sigma}_1 / \bar{\sigma}_3$	μ %	MC%	e	S %	MC%	e	S%	ϕ°	Remarks
A-72-1	2.50	4.10	1.61	0.39	2	2.52	5.25	29.6	.88	92.3	33.5	.95	96.2	25.8	Natural soil
72-2	3.34	5.91	2.57	1.43	4	2.30	4.5	[illegible]	.92	92.9	34.6	.95	99.8	23.6	" "
72-3	5.56	11.06	5.50	1.50	7	2.01	6.22	31.4	.86	100	31.1	.87	97.3	19.5	" "
A-73-1	4.27	6.11	2.31	1.62	4	2.77	6.37	36.5	1.02	97.9	29.3	.84	95.7	28.2	Reached with NaCl
73-2	4.15	6.20	2.04	1.97	4	3.08	5.0	33.0	1.06	95.4	32.0	.90	97.4	30.2	" " $KClO_3$
73-3	4.03	7.29	3.28	0.75	4	2.24	5.5	31.4	.86	99.3	33.7	.93	98.4	22.7	Natural sample
A-74-2	5.51	8.60	3.11	0.90	4	2.77	5.75	24.4	.67	99.3	22.6	.65	95.2	27.8	Leached with $CaCl_2$
A-75-3	2.28	3.85	1.57	0.42	2	2.47	4.75	33.3	1.00	92.2	38.2	1.07	97.6	24.7	$CaCl_2$ on rebound

NOTE:

1. For calculations of MC, e and S, the original volumes were determined by measurements. The length was measured twice and averaged. The cross section was determined from 2 measured diameters at right angles at 3 locations on the specimen (top, centre and bottom). The final volume after testing was measured by mercury immersion.

2. ϕ value based on zero intercept on shear axis.

3 All pressures in kg/cm²

TABLE A4

Results of Ion Determinations for Test Hole A

Sample	Ion, % of Total Flame				Exch. Cap.	Remarks
	Na	Ca	Mg	K	Me/100 gm.	
A-72	5.0	63.6	26.0	5.4	27.4	Natural soil, before testing
A-72-1	4.5	65.6	25.4	4.5	26.4	" " , after testing
A-72-2	4.8	67.2	24.1	3.9	32.3	" " " "
A-72-3	5.5	67.0	22.6	4.9	28.1	" " " "
A-73-1	61.6	21.2	10.0	7.2	25.0	After NaCl leach, before testing
A-73-1	80.5	12.2	5.1	2.2	25.2	" " " after testing
A-73-2	1.8	28.5	13.5	56.2	23.3	" $KClO_3$ " " "
A-73-3	3.9	69.5	21.8	4.8	26.0	Natural soil, after test
A-74-2	4.2	70.0	22.4	3.4	20.2	Natural soil, before test
A-74-2	0.9	98.0	0.1	1.0	18.2	After $CaCl_2$ leach, after test
A-75-3	4.9	67.0	22.6	5.5	29.8	Natural soil, before test
A-75-3	4.9	67.8	22.2	5.1	30,4	$CaCl_2$ on Rebound, top of sample
A-75-3	1.8	82.0	10.3	5.9	29.4	$CaCl_2$ on Rebound, bottom of sample.

Determinations are not corrected for cations held in pore water.

TABLE A5

Comparison of Leached Samples from 73' and 74'

	Property	Most	Desirability		Least
1	$(\sigma_1 - \sigma_3)$ Kg/cm^2	Ca 5.51	Na 4.27	K 4.15	nat 4.03
2	Kg/cm^2	Ca 8.60	nat 7.29	Na 6,41	K 6.20
3	Kg/cm^2	nat 3.28	Ca 3.11	Na 2.31	K 2.04
4	Pp Kg/cm^2	nat 0.75	Ca 0.90	Na 1.62	K 1.97
5		nat 2.24	Ca 2.77	Na 2.77	K 3.08
6	%	K 5.0	nat 5.5	Ca 5.75	Na 6.37
7	Final M.C. %	Ca 22.6	K 29.3	K 32.0	nat 33.7
8	Final e	Ca 0.65	Na 0.84	K 0.90	nat 0.93
9	Final S	nat 98.4	K 97.4	Na 95.7	nat 95.4
10	degrees	K 30.2	Na 28.2	Ca 27.8	nat 22.7

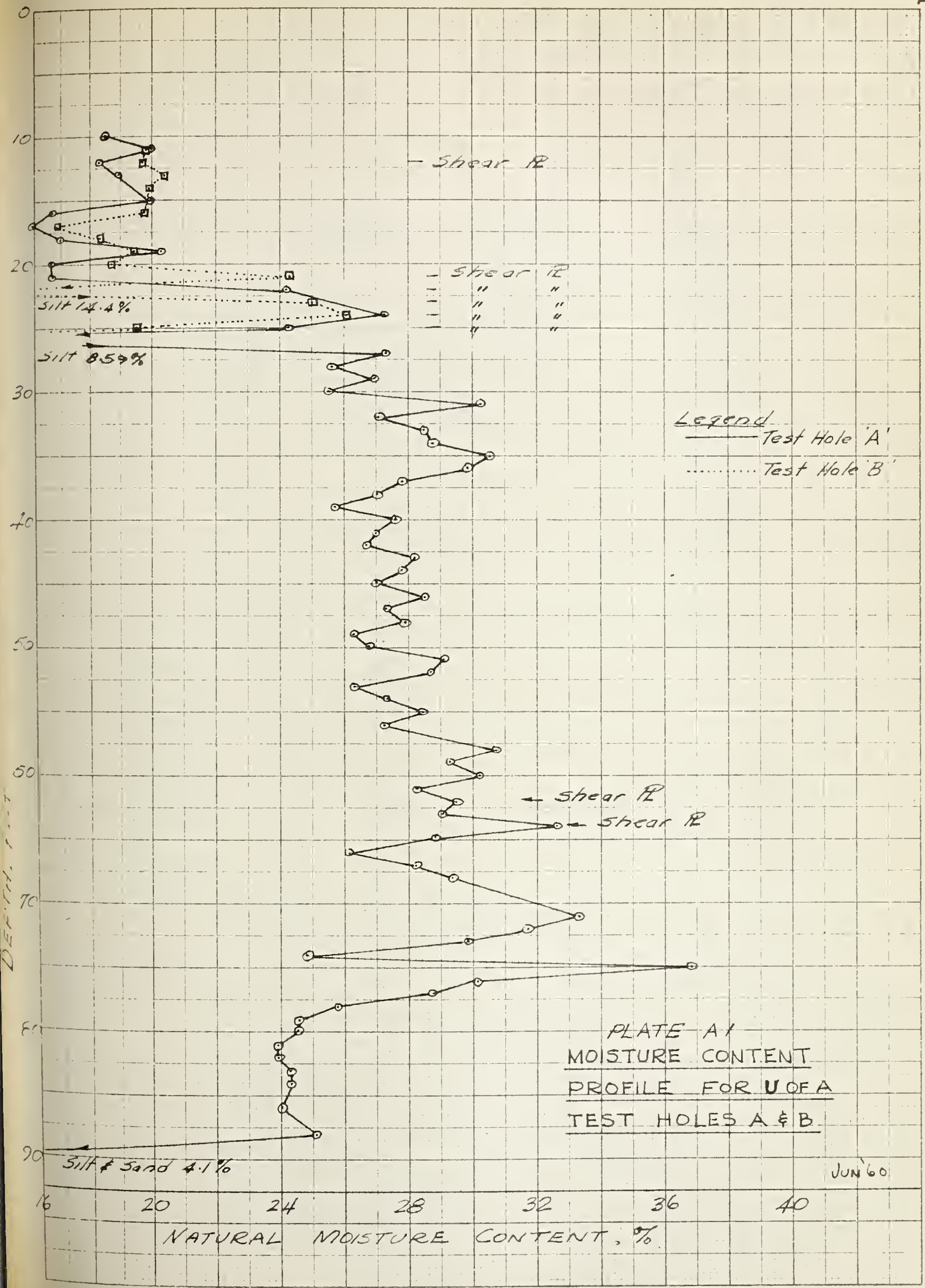
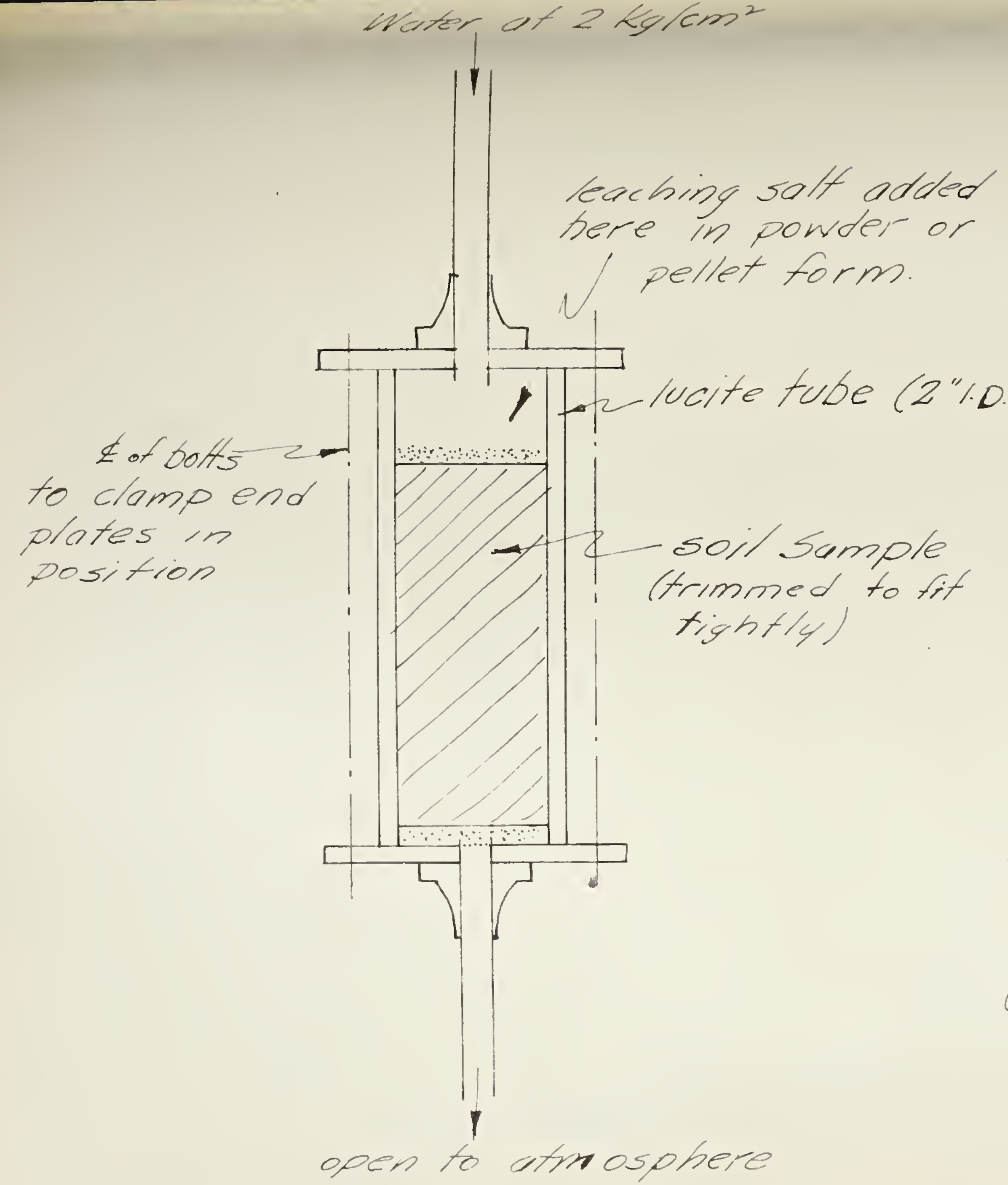


PLATE A1

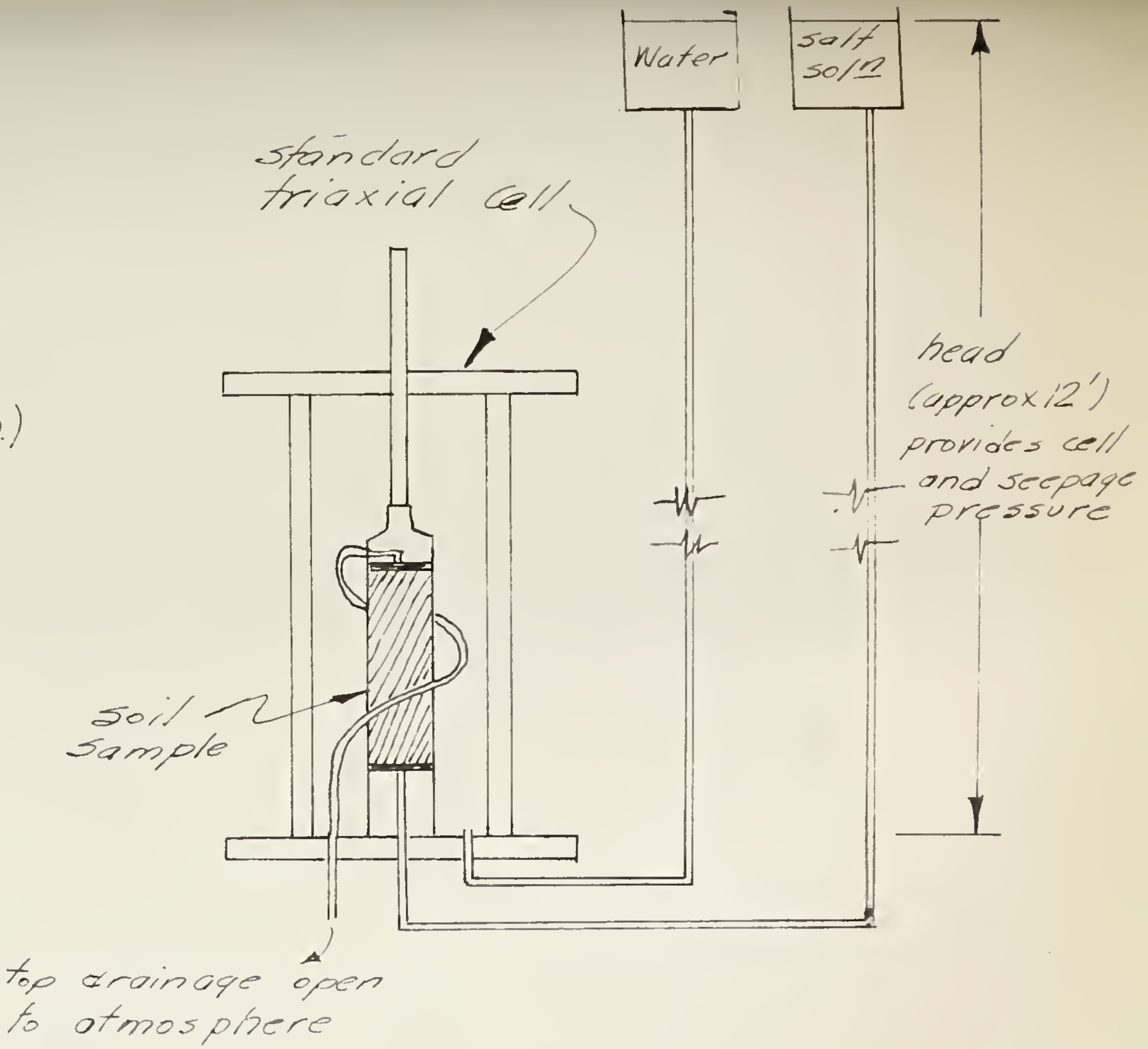
MOISTURE CONTENT PROFILE FOR U OF A TEST HOLES A & B

JUN '60

PLATE A1


Water at 2 kg/cm²
leaching salt added here in powder or pellet form.
lucite tube (2"I.D.)
℄ of bolts to clamp end plates in position
soil Sample (trimmed to fit tightly)
open to atmosphere

(a) SCHEMATIC DIAGRAM OF LEACHING TUBE


Water
salt sol$^{\underline{n}}$
standard triaxial cell
head (approx 12') provides cell and seepage pressure
soil Sample
top drainage open to atmosphere

(b) SCHEMATIC DIAGRAM OF LEACHING WITH TRIAXIAL CELL

PLATE A 2 LEACHING UNDISTURBED SAMPLES

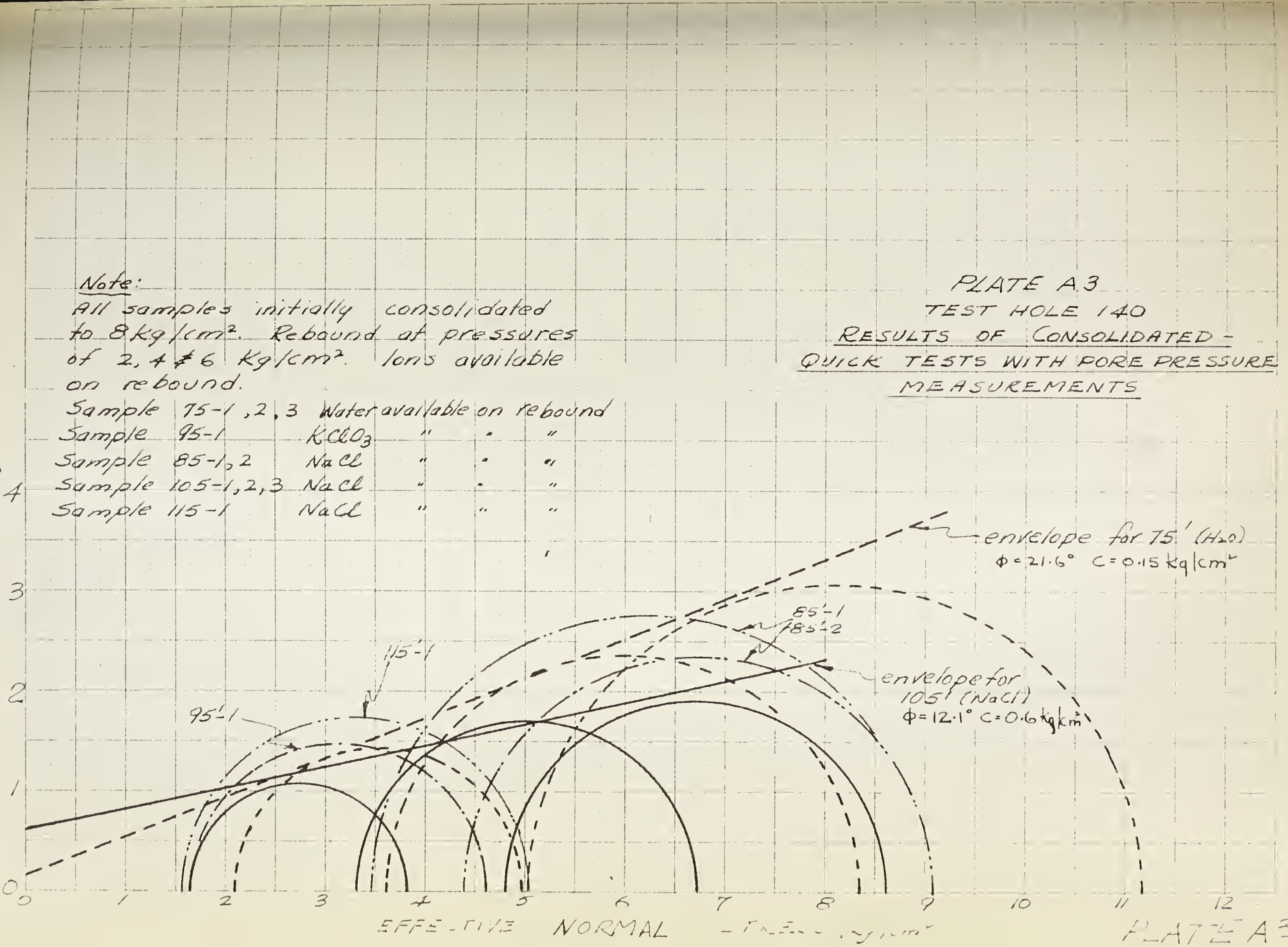


PLATE A3

TEST HOLE 140

RESULTS OF CONSOLIDATED - QUICK TESTS WITH PORE PRESSURE MEASUREMENTS

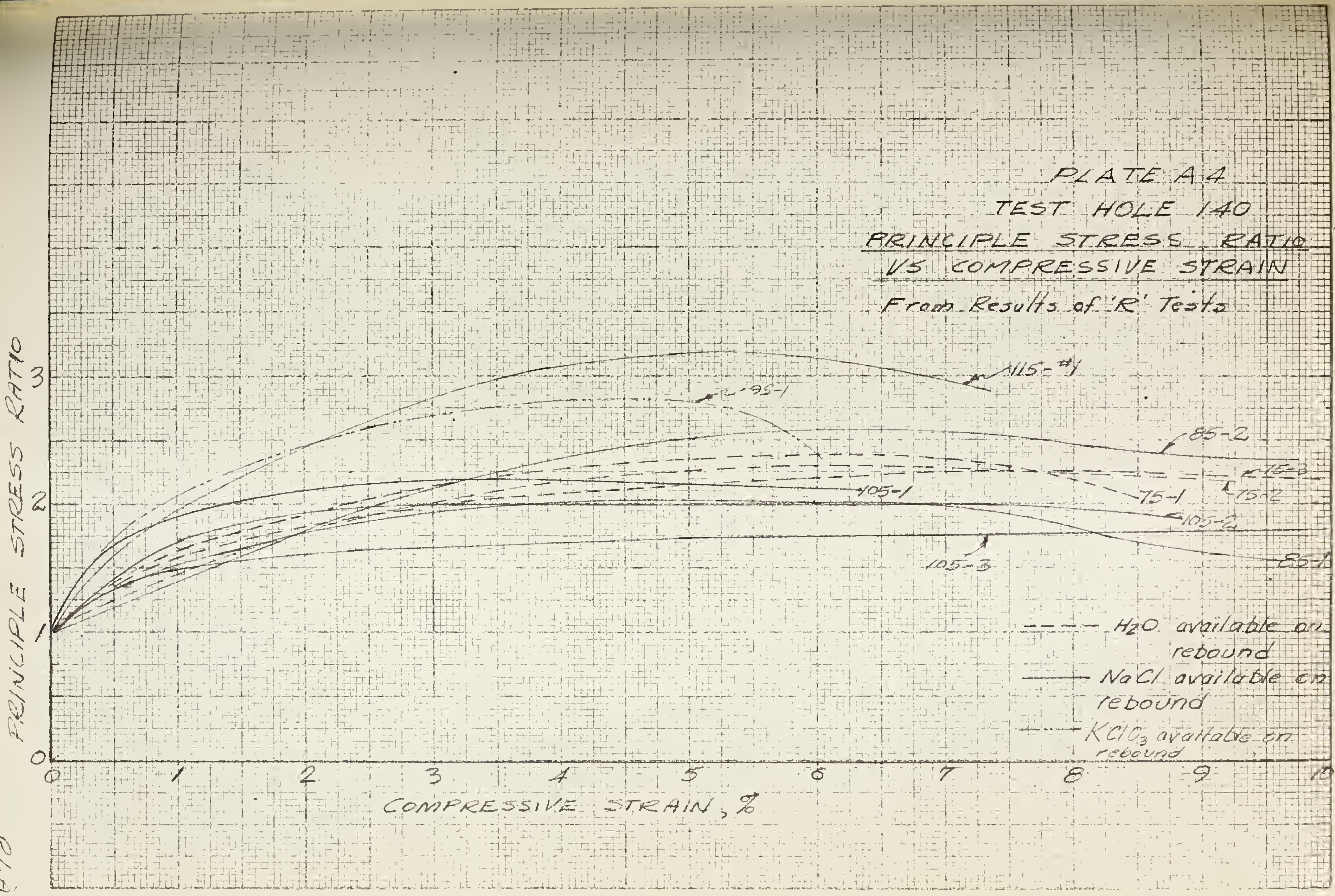
Note:
All samples initially consolidated to 8 kg/cm². Rebound at pressures of 2, 4 & 6 kg/cm². Ions available on rebound.

Sample 75-1, 2, 3 Water available on rebound
Sample 95-1 $KClO_3$ " " "
Sample 85-1, 2 NaCl " " "
Sample 105-1, 2, 3 NaCl " " "
Sample 115-1 NaCl " " "

envelope for 75' (H_2O)
$\phi = 21.6°$ $C = 0.15$ kg/cm²

envelope for 105' (NaCl)
$\phi = 12.1°$ $C = 0.6$ kg/cm²

EFFECTIVE SHEAR STRESS, kg/cm²
0
1
2
3
4
EFFECTIVE NORMAL STRESS, kg/cm²
0
1
2
3
4
5
6
7
8
9
10
11
12
95'-1
115'-1
85'-1
85'-2



THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO
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PLATE A4

PLATE 4

Notes:

(1) Specimens 72-1,2,3 standard 'R' Test, no ion exchange.

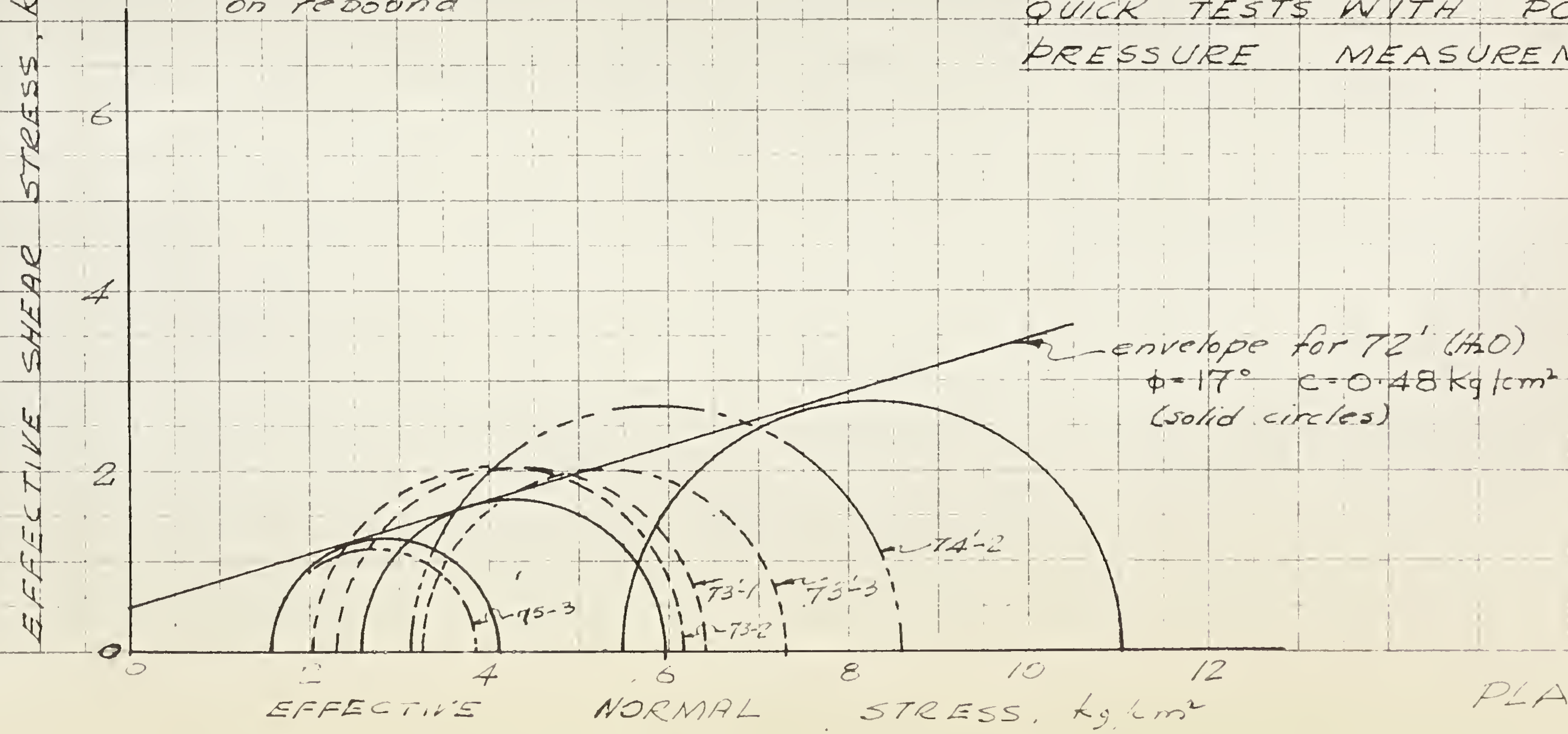
(2) Specimen 73-1 leached with NaCl
" 73-2 " " $KClO_3$
" 73-3 natural test

(3) Specimen 74-2 leached with $CaCl_2$

(4) Specimen 75-3 $CaCl_2$ available on rebound

PLATE A5
TEST HOLE 'A'

RESULTS OF CONSOLIDATED-QUICK TESTS WITH PORE PRESSURE MEASUREMENTS



THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO

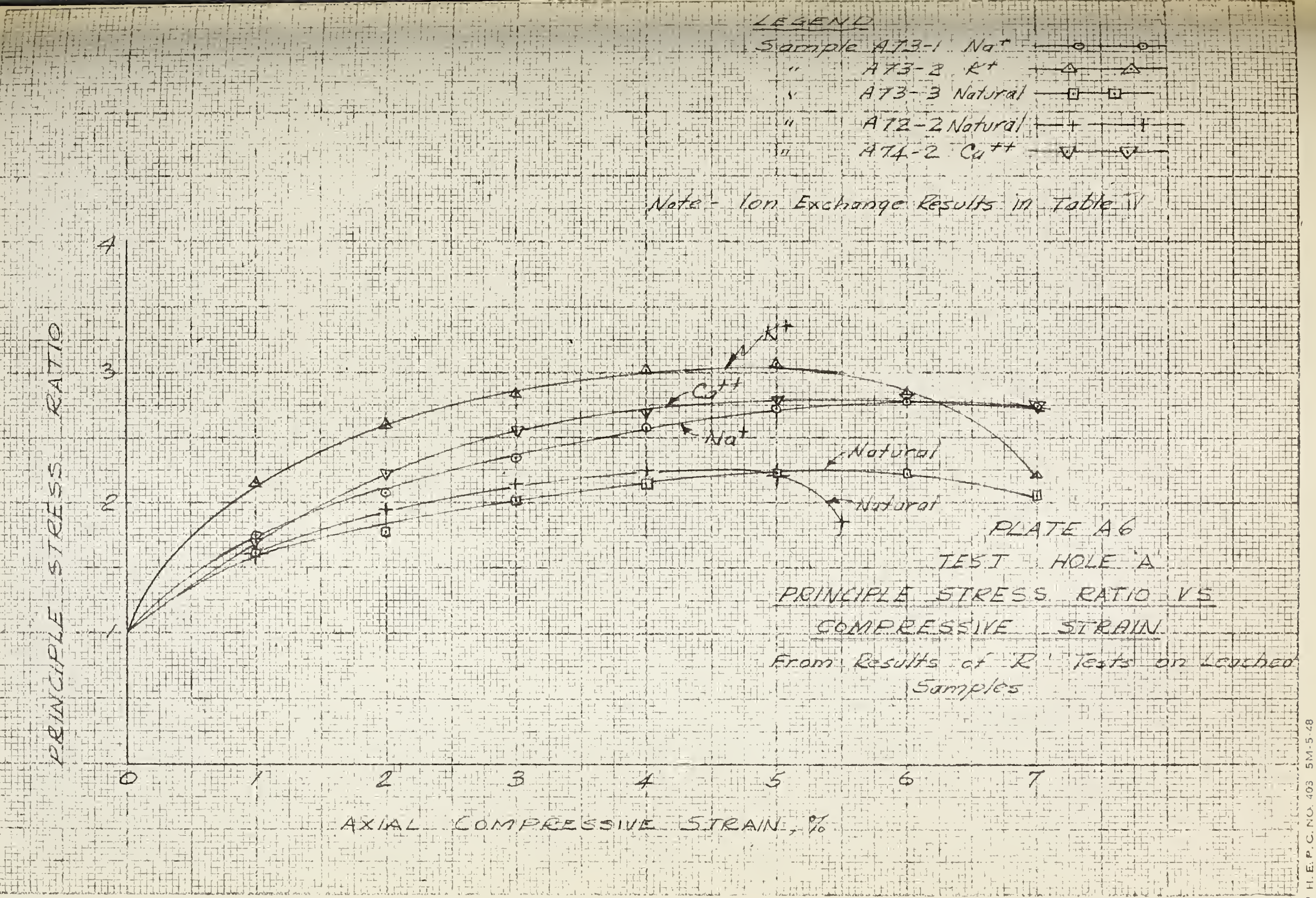


PLATE A6
TEST HOLE A
PRINCIPLE STRESS RATIO VS
COMPRESSIVE STRAIN
From Results of 'R' Tests on Leached Samples

PLATE A6

PLATE 6

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PLATE A7

ION EXCHANGE RESULTS

% of Exchange Positions

Sample	Na	Ca	Mg	K	Exch Cap m.e./100gm	
#1	1·8	67·7	25·4	5·1	27·5	natural Soil
#2	2·3	64·4	28·8	4·5	26·4	KCl available on rebound
#3	1·8	73·2	20·2	4·8	26·8	$CaCl_2$ " " "

Note Values corrected for Soluble Salts

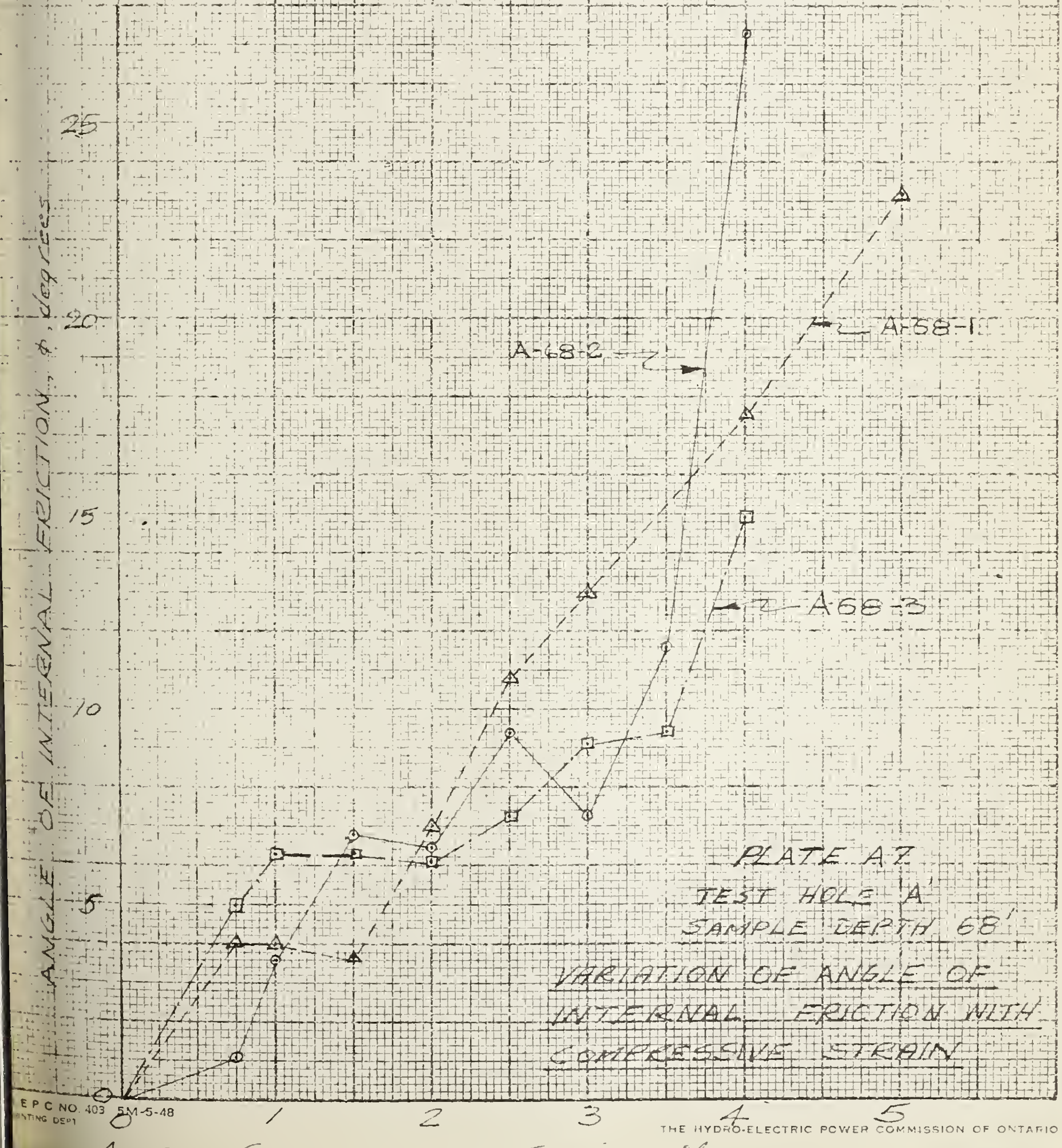

PLATE A7
TEST HOLE A
SAMPLE DEPTH 68'
VARIATION OF ANGLE OF INTERNAL FRICTION WITH COMPRESSIVE STRAIN

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THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO

PLATE A7

PLATE A8

ION EXCHANGE RESULTS

% Exchangable Positions

SAMPLE	Na	Ca	Mg	K	Exchange Cap m.e./100gm.
#1	1.8	67.7	25.4	5.1	27.5
#2	2.3	64.4	25.8	4.5	26.4
#3	1.8	73.2	20.2	4.8	26.8

Note: Values corrected for Soluble Salts

PLATE A8
TEST HOLE 'A'
SAMPLE DEPTH 68'
VARIATION OF COHESION
WITH AXIAL STRAIN

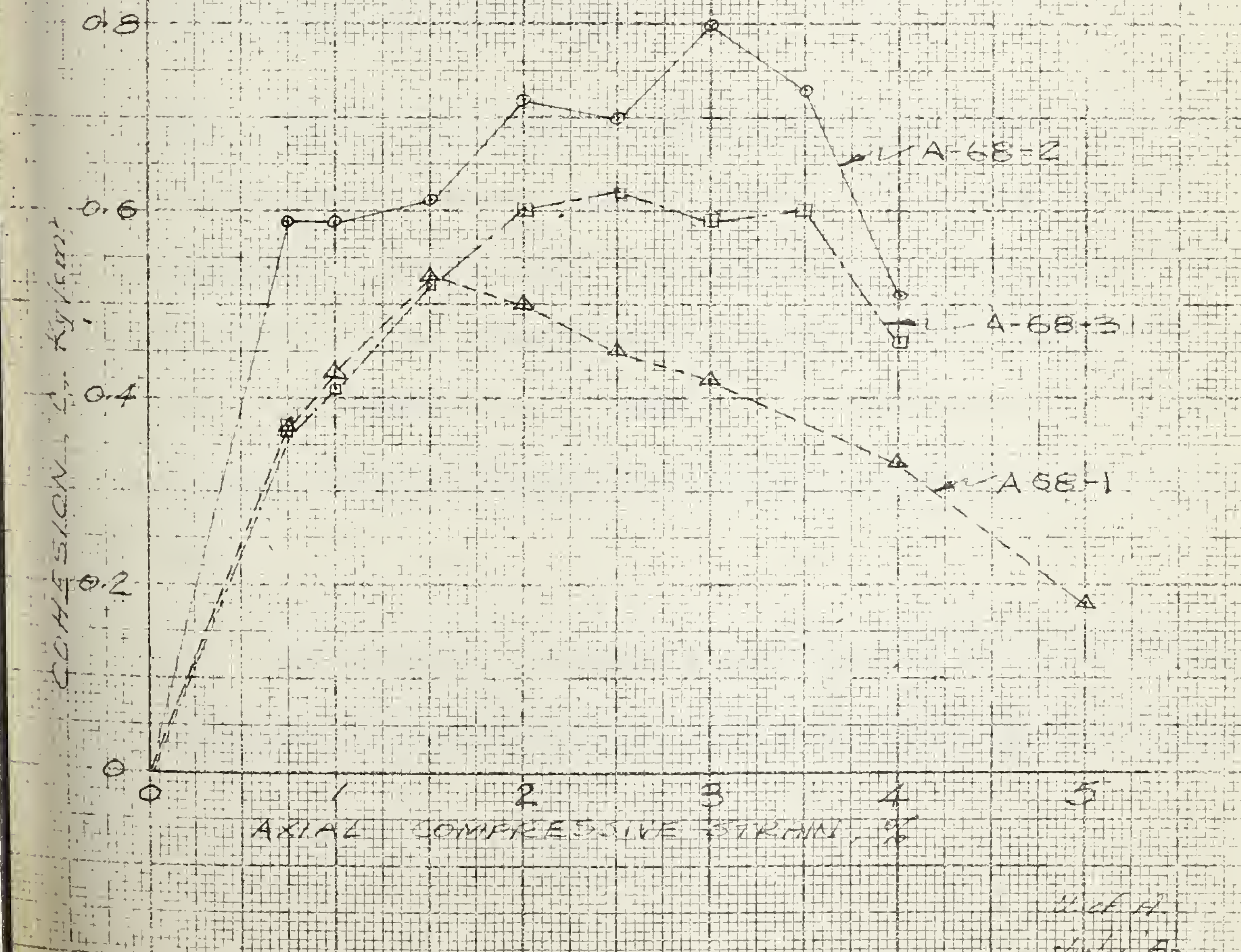

PLATE A8

SECOND PROGRESS REPORT ON EFFECTS OF CATIONS ON THE SHEAR STRENGTH OF AN ALBERTA CLAY SHALE

A2.1 It has been demonstrated by recent research that the nature of the adsorbed ion on the clay particles of a soil does affect the engineering properties. Recent work at Alberta by P. A. Thomson and A. B. Hamilton has produced some results dealing with the Atterberg limits and the consolidation characteristics of soils with a known and predetermined adsorbed ion complex. The specific problem, but still of broad scope, of the author is to investigate the influence of the adsorbed ion on the shear strength of Alberta clays.

A2.2 There are two major considerations of this work. One of these is to determine whether a loss of strength in field problems may be attributable, at least in part, to a natural change in the adsorbed ion. The change in stress-strain characteristics is a part of this work. The other major consideration is whether improvements in soil strength can be brought about by changing the adsorbed ion complex by chemical additives.

A2.3 The technique of preparing homionic soils (i.e. a soil having all one type of cation adsorbed on the clay minerals) has been mastered.

The technique of substituting portions of one ion for another on the clay complex has also been successfully established. The problem of producing samples for triaxial shear testing has also been established. In general, it may be said that the difficulties in sample preparation and testing have been overcome.

A2.4 The program is to determine the Mohr envelopes for 4 homionic modifications and the natural soil in a remoulded state. These results augmented by hydrometer grain size analyses and flame photometer cation determinations should shed some light on the problem of cation effects on shear strength.

A2.5 The program as of 30 June is about 40% complete and progress should speed up to some extent since techniques have become established. There is, at present, insufficient data on which to base any predictions of final results.

THIRD PROGRESS REPORT ON EFFECTS OF CATIONS ON THE SHEAR STRENGTH OF AN ALBERTA CLAY SHALE

A3.1 The original problem is to determine whether a loss of shear strength in a clay soil under field conditions may be partly due to a natural change in the adsorbed ion complex. Also of major importance is change in stress-strain characteristics and whether, improvements in soil strength can be brought about, either directly or indirectly, by chemical additives as they affect the adsorbed ion complex. A quantitative measure of strength gains or losses of a soil mass, even if approximate, would be of value. It is also hoped that the basic problem of the shear strength of cohesive soils will be made a little clearer by this work.

A3.2 As was indicated by the progress report of early July 1961, the difficulties of sample preparation and testing have been overcome. The work of the remainder of the summer added to the knowledge of sample preparation in that a few more problems arose and solutions were found. The technique of preparing a sample having a predetermined adsorbed ion complex has been reasonably well mastered.

A3.3 To date thirty-six consolidated-quick triaxial tests with pore

pressure measurements have been carried out in addition to ten hydrometer analyses with associated specific gravity determinations and nearly sixty cation exchange and flame photometer tests. A small amount of work has to be finished to complete this phase of the research. The mass of data observed has not been completely assimilated at this date. However, some trends are clearly evident and a thorough study of the data should yield some good results. The two most noteable results are the effective angles of internal friction and stress-strain characteristics. The attached plots of the Mohr envelopes, Plates A9 and A10, shows the angle of internal friction to be in the order of twenty two degrees for the natural clay and homionic calcium, potassium and magnesium states. For the sodium clay the angle is about eight degrees; a considerable reduction. For the four former states failure occurred at about ten percent strain whereas for sodium clay, failure occurred at about three percent strain. There is no doubt that the presence of sodium in the adsorbed ion complex affects both stress and strain.

A3.4 It is strongly recommended that this work continue. Before a reasonably comprehensive theory to explain this phenomena can be advanced other ions of various valence and hydrated ionic radii must be investigated. More quantitative information must be gathered on the change in strength with the amount of sodium adsorbed on the clay

complex. This would allow estimates of the amounts of sodium that can be tolerated in the adsorbed complex before strength loss becomes significant. Work should also be carried out to assess the amount of clay sizes that must be present before cation exchange becomes significant. Consolidation characteristics should also receive attention.

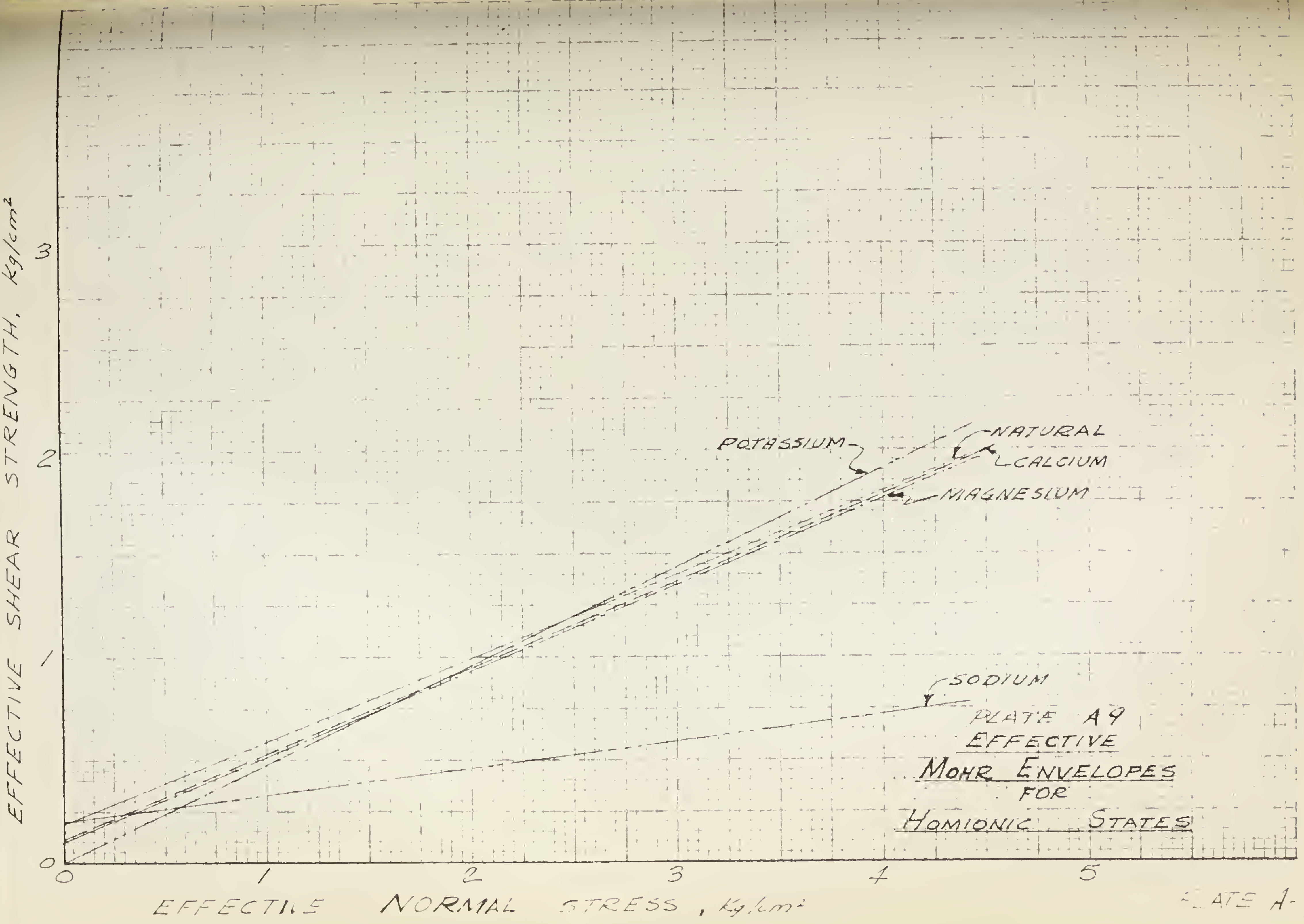

PLATE A9
EFFECTIVE
MOHR ENVELOPES
FOR
HOMIONIC STATES

PLATE A-9

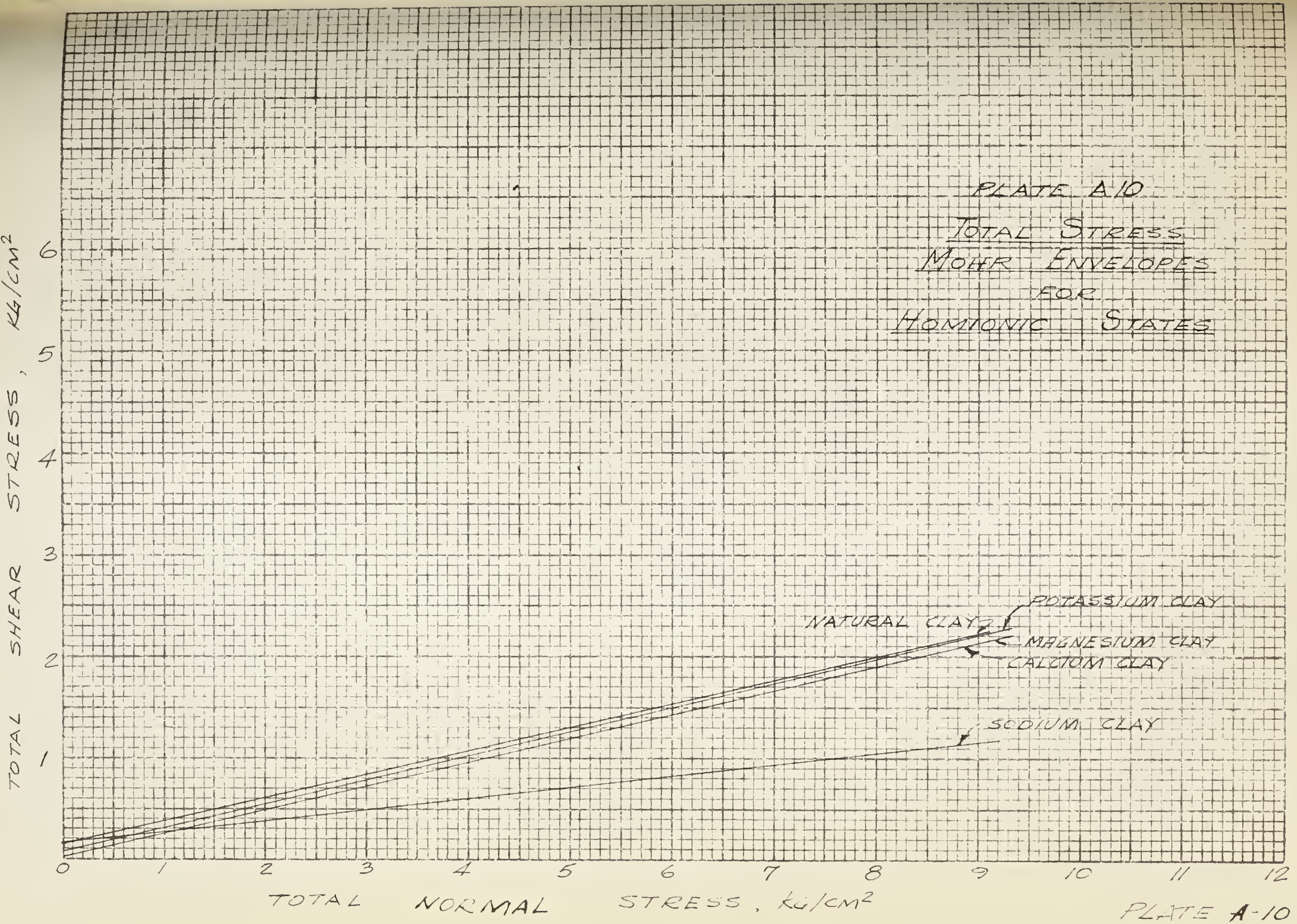
PLATE A10
TOTAL STRESS
MOHR ENVELOPES
FOR
HOMIONIC STATES
POTASSIUM CLAY
NATURAL CLAY
MAGNESIUM CLAY
CALCIUM CLAY
SODIUM CLAY
TOTAL SHEAR STRESS, KG/CM²
1
2
3
4
5
6
0
1
2
3
4
5
6
7
8
9
10
11
12
TOTAL NORMAL STRESS, kg/cm²

PLATE A-10

APPENDIX B

CLAY MINERALOGY

APPENDIX B

CLAY MINERALOGY

Introduction

B.1 Soils classified as 'clays' generally contain a mixture of clay minerals and other rock forming compounds such as quartz. It is known that the type of clay mineral present, as well as the amount of clay sizes, exerts an influence on such soil properties as Atterberg limits and compressibility. It is, therefore, the purpose of this appendix to review briefly the main clay minerals, their makeup and the techniques available for their identification. A further reason for the inclusion of this appendix is that the concepts of soil physics used to explain the behaviour of fine grained soils require a limited knowledge of clay mineralogy. The material herein is drawn largely from Grim (20)* to which the reader is referred for greater detail.

B.2 Prior to about 1925, when the first X-Ray difraction studies were made, chemical analyses were about the only investigations that had been made on clay minerals and these had shown that aluminum, silica, water, iron and the alkaline earths (sodium, calcium, potassium

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* Numbers in parentheses refer to references in the bibliography of this thesis.

and magnesium) were present but no idea was gained of how these were combined in the lattice structure.

B.3 Subsequent studies using X-Ray diffraction and other techniques established that the argillaceous materials are composed of very small particles (1μ or less in diameter) and that they are predominantly crystalline, i.e., the atoms are arranged in definite geometric patterns. They can generally be classed into a small group known as clay minerals.

B.4 Most of the clay minerals have sheet or layered structures but some have elongated or tubular structures or fibrous structures. Argillaceous material may be considered as essentially made up of extremely small particles, each one of which is either a book of sheet like units or a bundle of tubes or fibres. Individual soils (clays) may contain a mixture of book like units or a mixture of books and bundles along with other components.

Structure and Composition

B.5 The fundamental building blocks of the clay minerals are the silica tetrahedron and the alumina octahedron. These two units are described as follows:

(a) The silica tetrahedron consists of a central silica atom surrounded by four oxygens or hydroxyls in the form of a pyramid. See Figure B1.

These tetrahedrons combine in a sheet so that the oxygens of the bases are in a common plane and each oxygen (or Hydroxyl) belongs to two tetrahedra. The sheet is continuous in the a and b directions (horizontally). It can be considered as a layer of hydroxyls, a layer of silicons and a layer of hydroxyls or oxygens. In a clay mineral the thickness of this sheet is 4.93Å. The space available for the ion in tetrahedral coordination is in the order of 0.55Å which means that an ion with an ionic radius greater than this will distort the tetrahedral configuration.

(b) The octahedron may be considered as an aluminum surrounded by 6 hydroxyls in octahedral coordination. See Figure B2. It is a sheet structure tied together at the apices and is continuous in the a and b directions. It may be viewed as two layers of hydroxyls enclosing a layer of aluminums. The thickness of this unit in a clay mineral structure is 5.05Å. The space available within the lattice is 0.61Å hence distortion of the octahedron will occur if an ion of larger radius occupies the central position.

B.6 In these units it has been inferred that silicon is the central ion in the tetrahedral unit and aluminum in the octahedral sheet. These two atoms are not the only atoms that can occupy these positions. Aluminum can substitute for silicon in the tetrahedrons and such minerals as iron (both the ferric and ferrous states), magnesium, zinc, and

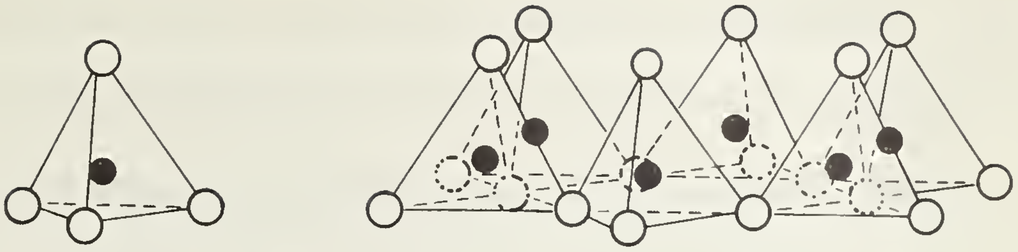

FIGURE B1 Diagrammatic sketch of a silica tetrahedron and the tetrahedrons arranged in a sheet structure. (After Grim, Reference 20)

○ = Oxygens and ● = Silicons

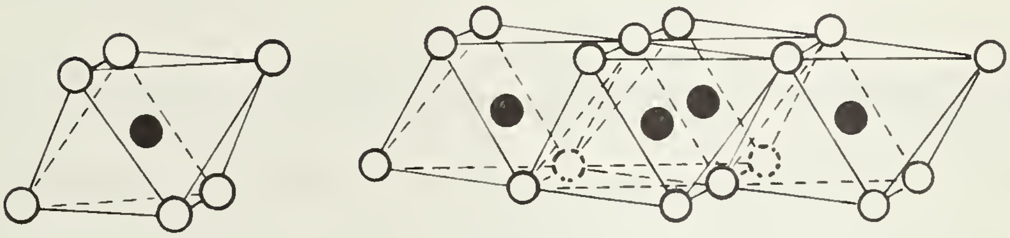

FIGURE B2 Diagrammatic sketch of an octahedral unit and the octahedrons arranged in a sheet structure. (After Grim, Reference 20)

○ = Hydroxyls ● = Aluminums, Magnesiums, etc.

chromium can substitute for aluminum in the octahedral units. Such substitution is referred to as isomorphous substitution. If the ionic radius of the substituting ion causes distortion the result is a curving of the sheet which gives rise to the tubular clay minerals.

B.7 In the octahedral sheet it is not necessary that all the positions within the unit be occupied. If all are filled it is referred to as tri-octahedral coordination and if two-thirds of the positions are filled it is referred to as dioctahedral coordination. Two special names are given to the octahdral sheets; when aluminum is the sole central atom it is referred to as "gibbsite"; when the sole central atom is magnesium it is known as "brucite".

B.8 The clay minerals have been generally considered to comprise three broad categories based on their general structure. These classes are kaolinite, montmorillonite and illite. There are, however, other clay minerals which are more conveniently considered separately. Three of these, namely chlorite, vermiculite and glauconite, are grouped under the heading 'Other Clay Minerals". The last general category considered is the mixed layer minerals. Each of these groups is discussed in the following sections.

Kaolinite

B.9 The members of kaolinite family are kaolinite, halloysite, metahalloysite, dickite and nacrite.

B.10 The structural unit of kaolinite is comprised of an octahedral layer with a superimposed tetrahedral layer intergrown such that the tips of the tetrahedrons and the apices of the octahedrons form a common layer. This unit is about 7Å thick and can extend indefinitely in the two directions perpendicular to the thickness. The mineral is composed of a stacking of such sheets with the sheets being held together by the relatively strong hydrogen bond. Many analyses have shown that there is very little substitution within the lattices and that only two-thirds of the central octrahedral positions are filled. Variation in the members of this group arises in the way in which the units are stacked and possibly in which of the octahedral positions are occupied.

B.11 The mineral halloysite is considered to consist of kaolinite layers separated from each other by a single molecular layer of water. Also the kaolinite layers are displaced randomly in the a and b directions. Transition to the metahalloysite (dehydrated) form takes place on loss of the interlayer water.

B.12 Electron micrographs show kaolinite to consist of well formed

six sided flakes. Surface dimensions are in the order of 0.3 to 4 microns and thicknesses .05 to 2 microns. Halloysite is considered to consist of elongate tubular particles having outside diameters of .04 to .19 microns and inside diameters of 0.02 to 0.1 microns. The tubes are overlapped thin flakes curled up due to the strain imposed on the octahedral layers and tetrahedral layers by their misfit. Interlayer bonds of a large number of successive sheets are not sufficient to overcome the imposed strains.

Montmorillonite

B.13 The basic montmorillonite layer is composed of an octahedral sheet enclosed by two tetrahedral sheets such that the tips of the tetrahedrons point inward towards the octahedrons. A common layer of oxygens is formed by the tips of the tetrahedrons and apices of the octahedrons. This unit is continuous in the a and b directions and is stacked in the c direction. In this stacking, the oxygen layers forming the bases of tetrahedral layers are adjacent to each other resulting in a weak bond and excellent cleavage. An outstanding feature of this mineral is the ability of water molecules and cations to enter between the unit layers causing expansion of the lattice in the c direction. In the instance of adsorbed cations, the distance between unit layers may be a function of the hydrated radius of the cation. If sodium is the adsorbed

cation the unit layers may separate completely but with adsorbed calcium separation is not complete.

B.14 There is always substitution within the lattice of aluminum for silicon in the tetrahedrons and of magnesium, iron, zinc, nickel, etc. for aluminum in the octahedrons. In the tetrahedral sheet substitution appears to be limited to some 15 percent or less. In the octahedral sheet, however, substitution may be partial to complete. Total replacement of aluminum by magnesium yields the mineral saponite, by zinc yields sauconite and by iron yields nontronite. Since the space available in octahedral coordination if o.55Å and the iron atom has a radius of 0.67Å, the lattice of nontronite is distorted to form elongated laths or needles.

B.15 A further characteristic of this mineral is that the lattice is always electrostatically unbalanced having a net negative charge due to the substitutions noted above. Unbalance in one sheet may be partially compensated for by substitutions in the other sheet. For example, the charge arising from substitution of trivalent aluminum for quadrivalent silicon may be partially offset by filling a few more of the octahedral positions. The remaining net charge deficiency is balanced by exchangeable ions. The seat of the charge deficiency is in the octahedral layer.

B.16 Electron micrographs of the montmorillonites reveal irregular, fluffy masses of extremely small particles. Some particles appear to be 0.002 microns thick and in the order of 10 to 100 times this in areal dimensions. Sodium montmorillonite appears as a continuous, filmy nebulous assemblage whereas calcium montmorillonite appears as irregular aggregates. The elongated lath shapes, flakes or needles of nontronite are clearly visible.

Illites or Hydrous Micas

B.17 The structural unit is the same as that of montmorillonite except that some of the silicons are always replaced by aluminums in the tetrahedral layer and this charge deficiency is balanced by adsorbed potassium ions. These units extend indefinitely in the a and b directions and are stacked in the c direction. The structural units are tied together by the adsorbed potassium ions between the layers where their radius is such that they just fit into the hexagonal perforations in the oxygen layer that forms the bases of the tetrahedrons. The adsorption of potassium in this manner is termed potassium fixation. They are very difficult to remove by normal laboratory leaching but are exchanged by weathering processes. The removal of potassium ions from the interlayers of illites is referred to as depotassification and is one mechanism for the conversion of illites to montmorillonites.

B.18 There appears to be a trend to favour the term 'hydrous micas' in place of 'illite' in the general classification due to the fact that this mineral is a gradational stage between mica as one end member and montmorillonite as the other end member. The transition appears to be a gradual replacement of the potassium by calcium, magnesium or hydrogen ions. These latter ions are too large to fit into the perforations of the oxygen layer and the mineral becomes montmorillonite. Also a factor in the transition is a decrease in the substitution of silicon by aluminum in the tetrahedral layer.

B.19 A seat of the negative charge deficiency in the illites is in the tetrahedral layers as opposed to the octahedral layers in montmorillonite. The seat of the charge being close to the surface of the unit layers allows a stronger bond between the mineral and the adsorbed cations. The fixation of potassium binding the layers together prevents the entry of polar ions hence the mineral is non-expanding.

B.20 Electron micrographs of illites show small flakes commonly grouped together. The thinnest flakes appear to be in the order of 30Å and 0.1 to 0.3 microns in diameter. The outlines are irregular but often well defined. The particles are larger, thicker and better defined than montmorillonite.

Other Clay Minerals

B.21 The three clay minerals, namely chlorite, vermiculite and glauconite, discussed under this heading may be placed in one of the preceding categories. However, it appeared to add to the clarity of this brief discussion of the clay minerals to place them under a separate heading.

B.22 Chlorites consist of alternate mica-like and brucite layers; two of each forming the unit cell. The unit is continuous in the a and b directions, stacked in the c direction and has basal cleavage between the layers. The mica-like layers are generally trioctahedral and are electrically unbalanced by substitution of aluminum for silica. This charge deficiency is largely balanced by an excess charge in the brucite sheet due to substitution of aluminum for magnesium.

B.23 The vermiculite structure is an alternation of trioctahedral mica and 'n' double water layers, the latter occupying a definite space arrangement. The structure is unbalanced by substitution of trivalent aluminum for quadravalent silica. The net charge deficiency is satisfied by cation adsorption. It has an expanding lattice and high exchange capacity.

B.24 Glauconite is a dioctahedral illite with ferric and ferrous iron

and magnesium substituting for aluminum. The charge deficiency is usually satisfied by interlayer ions which may be sodium or calcium as well as potassium.

B.25 In order to aid in visualizing the crystal make up of clays some of the clay minerals are schematically represented on Figure B3.

Mixed Layer Minerals

B.26 The clay particles existing in a clay soil do not necessarily consist of pure clay minerals. The particles may be comprised of a stacking of two or more clay minerals that may be mixed in any of several ways. Such mixtures are referred to as Mixed Layer Minerals. There may be discrete interlayering of pure minerals with no preferred geometric particle to particle orientation. Such particles will cleave easily through such mineral layers as montmorillonite. Another mixture possibility is interstratification as a consequence of the fact that the layers of clay minerals are very similar. These mixtures are as stable as pure minerals. The interstratification may be regular (e.g. chlorite) or random.

B.27 In any given clay sample there may be a mixture of particles of pure clay minerals. If, for example, montmorillonite particles are present in pure form then the influence they exert will be in rough

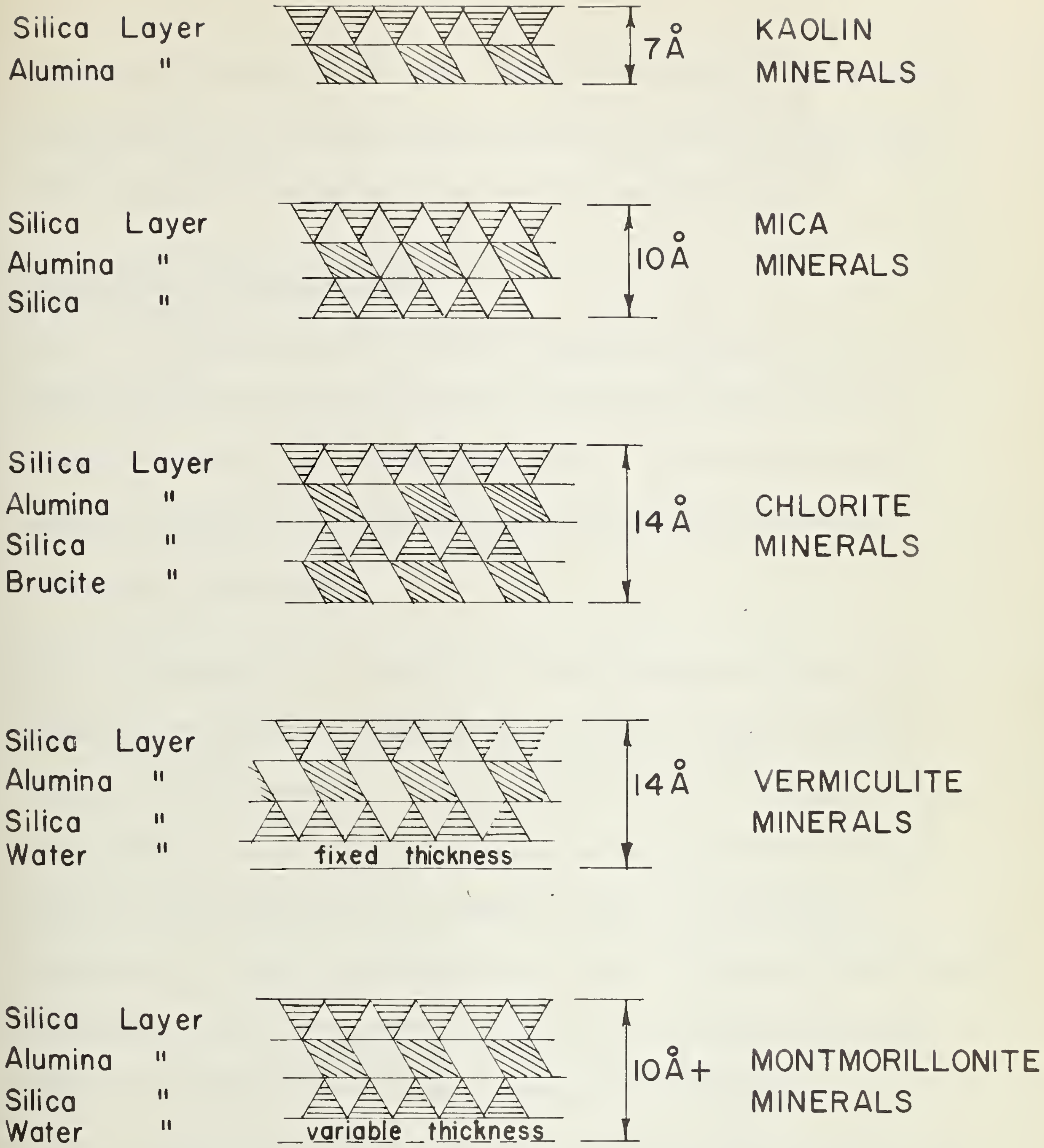

FIGURE B3 SCHEMATIC REPRESENTATION OF CLAY MINERALS

proportion to the amount present. This is in contrast to interlayering as mentioned in the previous paragraph where the same mineral may exert an influence far out of proportion to the amount present.

Identification of Clay Minerals

B.28 The identification of the clay minerals is considered to be of sufficient interest that the techniques available and a brief description of each are given in the following paragraphs. It is also felt that the engineer should be aware of processes involved should he wish the analysis of a clay sample.

B.29 The major tool available is X-Ray diffraction (7) which is based on the reflection of incident X-rays from the layers of atoms within the crystal structure. A reflected X-ray from the upper crystal surface will be reinforced by a reflected ray from the bottom surface when the crystal thickness is an integral number of wavelengths of incident rays. Hence, by measuring the angle of incidence of the X-rays and knowing their wavelength the thickness of the crystal can be determined. Each mineral has a unique thickness or 'd' spacing.

B.30 Dehydration curves reflect water loss from the mineral as the temperature is increased. Kaolinite, having little water adsorption, has little loss at temperatures below 300°C. The water of constitution

per unit weight is about twice as great as that of montmorillonite and passes off abruptly at about 500°C. This is distinctive for this mineral. Montmorillonite has high water loss at low temperatures due to a high water adsorption and the lattice water passes off over a large range of temperatures.

B.31 Differential Thermal Analysis is based on volatization of water absorbing energy, oxidation releasing energy and structural rearrangement of molecules usually releasing energy (34). One thermocouple is placed in the unknown sample and another placed in an inert material (e.g. alundum or nickel) such that the potentials are opposed. Both the unknown and the inert material are heated. There will be no heat flow if no reaction takes place; for an endothermic reaction the curve dips and for an exothermic reaction the curve rises. Identification is based on the fact that most clay minerals have an unique curve. The magnitude of the peaks or depressions is an approximate measure of the amount present.

B.32 Staining methods refer to the preferential adsorption of organic dyes and their study by optical microscopes.

B.33 Electron Microscopy uses beams of electrons instead of light and is capable of magnifications of 50,000 x (4). Its results have been mentioned in previous paragraphs.

B.34 Chemical analyses are self explanatory but have the serious drawback that the materials present can be determined but not their arrangement.

B.35 Generally any one of these techniques is insufficient to establish the identity of the clay minerals reliably and confirmatory or substantiating tests must be conducted, that is a combination of two or more tests are required to establish the identity of minerals in a sample.

B.36 A sample of a 'clay' soil to the Soil Mechanics engineer will usually contain a small amount of fine sand, a larger proportion of silt and the remainder, often 60% or less, clay sizes. The sand and silt is made up of roughly equidimensional particles of the parent rock merely decreased in size by the processes of physical weathering such as attrition, glaciation, etc. Typical minerals of this group are quartz, feldspars and plagioclases. The clay sizes contain particles of the parent rock finely dived as well as the clay minerals. The latter differ from parent material in that they have undergone chemical weathering to form new minerals which were not present originally. Separation can usually be accomplished on the basis of grain size except in the clay size ranges where sedimentation cannot separate clay minerals from finely ground parent rock.

Summary

B.37 Much factual data has been presented in this chapter but in summary the main points that an engineer should be aware of are the three main clay mineral structures and the characteristics of each group that have the greatest effect on the geotechnical properties.

B.38 The kaolinite sheet is comprised of a sheet of tetrahedrons and a sheet of octahedrons arranged in such a manner that the tips of the tetrahedrons and apices of the octahedrons form a common layer. There is very little isomorphous substitution within the lattices hence the capacity for adsorbing cations will be small.

B.39 The montmorillonite sheet is composed of an octahedral sheet enclosed by two tetrahedral layers. The main characteristics of this mineral are its excellent cleavage, its ability to expand in the 'c' direction and its high capacity to adsorb cations.

B.40 Illite is a gradational mineral whose properties are, in general, between those of kaolinite and montmorillonite. It is a non-expanding mineral due to the fact that adsorbed potassium ions tie the structural sheets together preventing ingress of water.

B.41 The clay soils seldom comprise a pure clay mineral but occur rather as mixtures of either an interlayer or mechanical type.

Combinations of two or more of the techniques available for the identification of clay minerals are generally required for mutual confirmation of the findings.

APPENDIX C

PHYSICAL CHEMISTRY

APPENDIX C

PHYSICAL CHEMISTRY

Introduction

C.1 The title of this appendix is perhaps a little misleading as it is intended to extract from the broad field of physical chemistry only those few definitions and concepts as are necessary to provide a background for the understanding of the soil physics to be presented elsewhere in this thesis. Physical chemistry is a study of the physical properties of matter. It deals with gases, liquids, solids, colloidal suspensions, energy relationships and physical transformations. The concepts briefly dealt with are the types of intermolecular bond, intermolecular forces and molecular polarization. Consideration of these follows a few basic definitions. The material for this appendix was drawn largely from Maron and Prutton (41)*.

Concentration

C.2 Two common means of expressing the concentration of a solution are molarity and normality. The former is the number of molecular

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* Numbers in parentheses refer to references in the bibliography of this thesis.

weights (moles) of a substance per litre of solution. Normality is the number of equivalent weights of a substance per litre of solution. An equivalent weight is the weight of a substance that will react with 1.008 grams of hydrogen.

Ionic Charge

C.3 The ionic charge is the net negative charge arising within the clay particle due to the electrostatic unbalance. The causes of this charge will be dealt with under Soil Physics.

Adsorption

C.4 Adsorption may be visualized as the physical holding of a particle on the surface of a body. It is defined as the change in concentration of the adsorbate at the surface or interface of the adsorbent.

Hydration

C.5 Hydration is the adsorption of the water molecule

Ionic Bond

C.6 Ionic bonding is the outright transfer of one or more electrons from one atom to another leading to the formation of ions held together by electrostatic attraction. This type of bonding is very strong and is exemplified by NaCl.

Covalent Bond

C.7 Covalent bonding accounts for such molecules as N_2, O_2, H_2, etc. and is the sharing of electrons in pairs, each electron pair corresponding to a single valence. If the bond arises from the sharing of electrons supplied by one atom only it is said to be coordinate covalent bonding. An example is the ammonium ion.

Hydrogen Bond

C.8 In certain compounds, a hydrogen already bound by two electrons to an atom may further coordinate two more electrons to form another bond. Physically it may be visualized as a proton situated between two oxygens. Due to the strong electronegativity of the oxygen atom, the electron cloud of the proton is shifted towards the nearer of the two oxygens, that is the oxygen tries to complete its electron rings by attracting electrons from the proton. This sets up an attraction between the proton and the one oxygen. Since the electron cloud of the hydrogen is distorted, the end remote from the nearer oxygen becomes more strongly positive hence there is a Coulombic attraction between the proton and the other oxygen.

C.9 Hydrogen bonding exists in the gaseous, liquid and solid states and is the bond that is responsible for holding the kaolinite sheets tightly together.

Electrical Polarization of Molecules

C.10 Any molecule is composed of positive nuclei and negative electrons. If a molecule is placed between two electrically charged plates, the anode attracts the electrons and the cathode attracts the nuclei, polarization of the molecule results and an electric dipole results. See Figure C1 (i). If the polarization disappears when the field is removed then the dipole is said to be induced and the molecule is said to be non-polar. In the induced state the centres of electric charge of the molecule of strength Z, are a distance 'l' apart. The electric moment formed is then of strength

$$\mu = Z.l$$

The magnitude of the electric moment, μ, is determined by the electric field strength, X, as follows

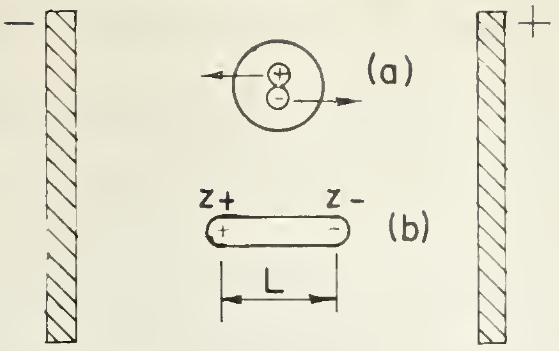
$$\mu = \alpha .X$$

where α is the polarizability of the given molecule. The magnitude of the induced polarization, P_i, is given by

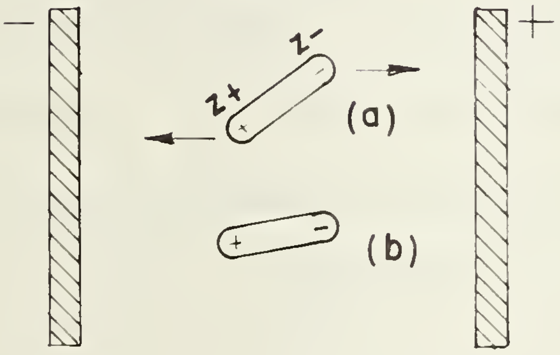
$$P_i = \frac{4}{3}\pi N \alpha$$

where N is Avogadro's number. Since N and α are temperature independent, then the magnitude of the induced polarization is also independent of temperature.

C.11 Some molecules, however, do show a variation of polarizability



(i) Formation of a dipole



(ii) Polarization of a molecule
with a permanent dipole

FIGURE C1 POLARIZATION OF A MOLECULE
IN AN ELECTRIC FIELD
(a) original state
(b) polarized state
(after Maron and Pruton, 41)

with temperature. Such molecules possess a permanent dipole which arises from the fact that their centres of positive and negative charge do not coincide. These molecules are termed polar. Normal orientation of dipole molecules is random due to thermal agitation. When placed in an electric field two affects occur with respect to the molecule. See Figure C1 (ii). The first is an orientation in the direction of the field and the second is the polarization of the molecule. The orientation polarization is equal to

$$P_0 = \frac{4}{3}\pi N\left(\frac{\mu^2}{3kT}\right)$$

where μ is the permanent dipole, k is Boltzman's constant and T is the absolute temperature. The induced polarization is as previously given, hence the total molar polarization is given by

$$P_t = P_i + P_0 = \frac{4}{3}\pi N\alpha + \frac{4}{3}\pi N\left(\frac{\mu^2}{3kT}\right)$$

For a given system, P_i will be a constant and P_0 will vary as the reciprocal of T hence the equation may be considered as follows

$$P_t = A + \frac{B}{T}$$

where A and B are constants. Since water is a polar fluid its effect in an adsorbed state should vary with temperature.

Intermolecular Forces

C.12 These are often referred to as Van der Waals forces and those leading to attraction are due to three actions. These are:

(a) The particle orientation, which is present only in molecules with a permanent dipole, arises due to the attractions of one dipole for another. If the dipole moment is μ, this force of attraction is

$$f_o = \left(\frac{4\mu^4}{kT}\right)\frac{1}{r^7}$$

where r is the distance of separation of the molecules.

(b) The presence of permanent dipoles causes induced dipoles in neighboring molecules. The interaction of the induced and permanent dipoles leads to an attractive force given by:

$$f_i = (12\alpha\mu^2)\frac{1}{r^7}$$

(c) Due to the vibrations of electron clouds with respect to the nuclei of atoms in a molecule a force of attraction arises which is given by:

$$f_d = \left(\frac{9h\upsilon\alpha^2}{2}\right)\frac{1}{r^7}$$

where h is Planck's constant and υ is a characteristic frequency of the charge distribution.

There is a repulsive force arising from the approach of electron atmospheres and nuclei of the atoms in one molecule with those in another. This force is of the form

$$f_r = \frac{B}{r^n}$$

where B is a constant for a given system and n varies from 10 to 13.

All these Van der Waals forces can be summed up in an equation of the form:

$$f = \frac{C}{r^7} - \frac{B}{r^n}$$

where C is a constant arising from the addition of the equations given in (a), (b) and (c). It is seen that these forces vary as, at least, the reciprocal of the seventh power of the distance between molecules. They are, therefore, rather small forces unless 'r' is very small.

Coulombic Forces

C.13 Coulombic forces operate over greater distances than the Van der Waals forces and arise from the electrical attraction and repulsion of two charges. The force of attraction is directly proportional to the product of the charges and inversely proportional to the square of the distance between them, (3) i.e.

$$F \propto \frac{e_a e_c}{(r_a + r_c)}$$

where e_a = charge on the anion

e_c = charge on the cation

r_a = radius of the anion

r_c = radius of the cation

Summary

C.14 In summarizing this chapter common terms have been defined. Polarization of molecules was briefly covered both for non-polar and polar

molecules. Van der Waals forces vary as the reciprocal of the seventh power of the distance between the molecules whereas Coulombic forces vary as the reciprocal of the square of the distance. The equations have been cited to indicate the variables that influence the various forces. The two most important characteristics of a particle being its polarizability, α , and its dipole moment, μ .

APPENDIX D

SOIL PHYSICS

APPENDIX D

SOIL PHYSICS

Introduction

D.1 Soil Physics is defined as a study of the mechanical behaviour or of the physical properties of soils (3)*. It has been generally directed toward the field of agriculture. Although this appendix is entitled soil physics, emphasis will be placed on cation exchange phenomena. The agronomist has been aware of the importance of cation exchange for over a century and has been intensively investigating it for more than fifty years. Engineers have only realized its value in the past thirty years and have only begun to appreciate its value or significance to any large extent in the past decade. It is the purpose of this appendix to provide a sufficient understanding of cation exchange that subsequent explanations of phenomena in the field of Soil Mechanics can be offered in terms of soil physics.

Surface Area

D.2 The forces associated with the surface area of a material are

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* Figures in Parentheses refer to the references in the bibliography of this thesis.

proportional to the area available. For large particles, that is the sand sizes, the surface forces are very small in comparison to body forces, for example, that of gravity. For sizes in the colloid range the reverse is true and the effect of gravity may be so small as to be neglected. In order to visualize the influence of surface area consider a cube with each side one centimeter long (3). The volume is one cubic centimeter and the surface area is 6 square centimeters. If this cube is now divided into cubes with a side one millimeter long, the particle is now in the coarse sand range and the original cube now yields an area of 60 square centimeters. Further subdivision to clay sizes, i.e. a dimension of 0.001 millimeter per side, gives a surface area of 60,000 square centimeters for the original cube. Final division to colloid sizes, i.e. to cubes with sides equal to 0.0001 millimeters, yields a surface area of 600,000 square centimeters for the original one cubic centimeter.

D.3 It should be readily apparent that the surface area varies with the shape of the particle. It is well known that a sphere has the smallest surface area per unit of volume. A single sphere of one cubic centimeter volume has a surface area of 4.8 square centimeters. A disc of the same volume with a thickness of 0.001 millimeters has a surface area of 20,000 square centimeters. Since clay particles are

known to be disc shaped, the surface area of a clay soil is very large in comparison to its mass.

D.4 As a direct consequence of the disc shape, the particle to particle relationships are profoundly influenced. One has only to consider the anology of a pack of cards and a collection of ball bearings. The particle to particle contact in the former case is enormously greater than in the latter, perhaps in the order of 90% to a very few percent. Particle shape must, therefore, exert a marked influence on the mechanical properties of systems composed of such particles. If, for example, a shear force is imposed on a system of flake shaped particles in a direction parallel to the flat surfaces, one particle can slide over another. If such a force is applied to a system of spheres then the spheres must roll apart or break. It would appear, then, that there is a fundamental difference in the mechanical behaviour of systems composed of flat shaped particles and systems of spherical shape.

D.5 The type of material also influences the forces associated with surface areas. The classic illustration of this point is the comparison of colloidal suspensions of gold and clay. Gold has practically no ion exchange capacity which indicates that the particles are chemically inert. There is no tendency for flocculation to occur hence suspensions

of gold are stable. On the other hand clays can have a high exchange capacity which denotes particles possessing considerable surface activity. Flocculation occurs easily unless the suspensions contain chemical additives to maintain stability. A more familiar illustration is a comparison of the Atterberg limits of such materials as rock flour and bentonite.

D.6 To summarize the factors influencing surface area, they are three in number. First, that of particle size; mere subdivision increasing the surface area per unit volume. Second, the particle shape effect with the disc yielding the greatest surface area per unit volume. Finally, the material comprising the particles. The surface forces of clays are enhanced to the point of exerting a profound influence on its mechanical properties. It is the expression of this type of phenomena that is being sought in the use of the 'activity'* in soil mechanics. If a clay sample contains 30% clay sizes and these are predominantly montmorillonite then, due to a large surface area, the plasticity index would be high and hence the activity would be high. If, on the other hand, most of the particles were kaolinite the surface area would be much less, the plasticity index lower and the activity lower. Activity, then,

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*Activity of a soil is defined as the Plasticity Index divided by the clay fraction where the clay fraction is the percentage by weight of particles finer than 2 microns.

may be considered as qualitative but approximate measure of surface area. The relative shapes and surface areas of the major clay minerals are sketched in Figure D1 which has been taken from Warkentin (66).

Causes and Relative Importance of Cation Exchange

D.7 In Appendix B it was mentioned that clay particles have net negative charges that are satisfied by adsorbed cations. The way in which the cations are adsorbed are three in number, each having some influence on the subsequent behaviour of the particle. Grim (20) gives the causes of cation exchange as:

(a) Broken bonds on the edges of the crystal linking oxygens or hydroxyls to the aluminums or silicons give rise to unsatisfied charges. These charges exist near the surface hence the adsorbed cation is more strongly held.

(b) Substitutions within the lattice structure of the mineral, e.g., trivalent aluminum for quadravalent silicon in the tetrahedrons or divalent ions (Mg, Fe, Zn, etc.) for aluminum in the octahedrons would give rise to unsatisfied charges. There appears to be a limit of substitution in the tetrahedrons to about 15% of the positions but there is no limit to substitution within the octahedrons. It follows, therefore, that the seat of the charge is in the octahedral layer hence the adsorbed cations

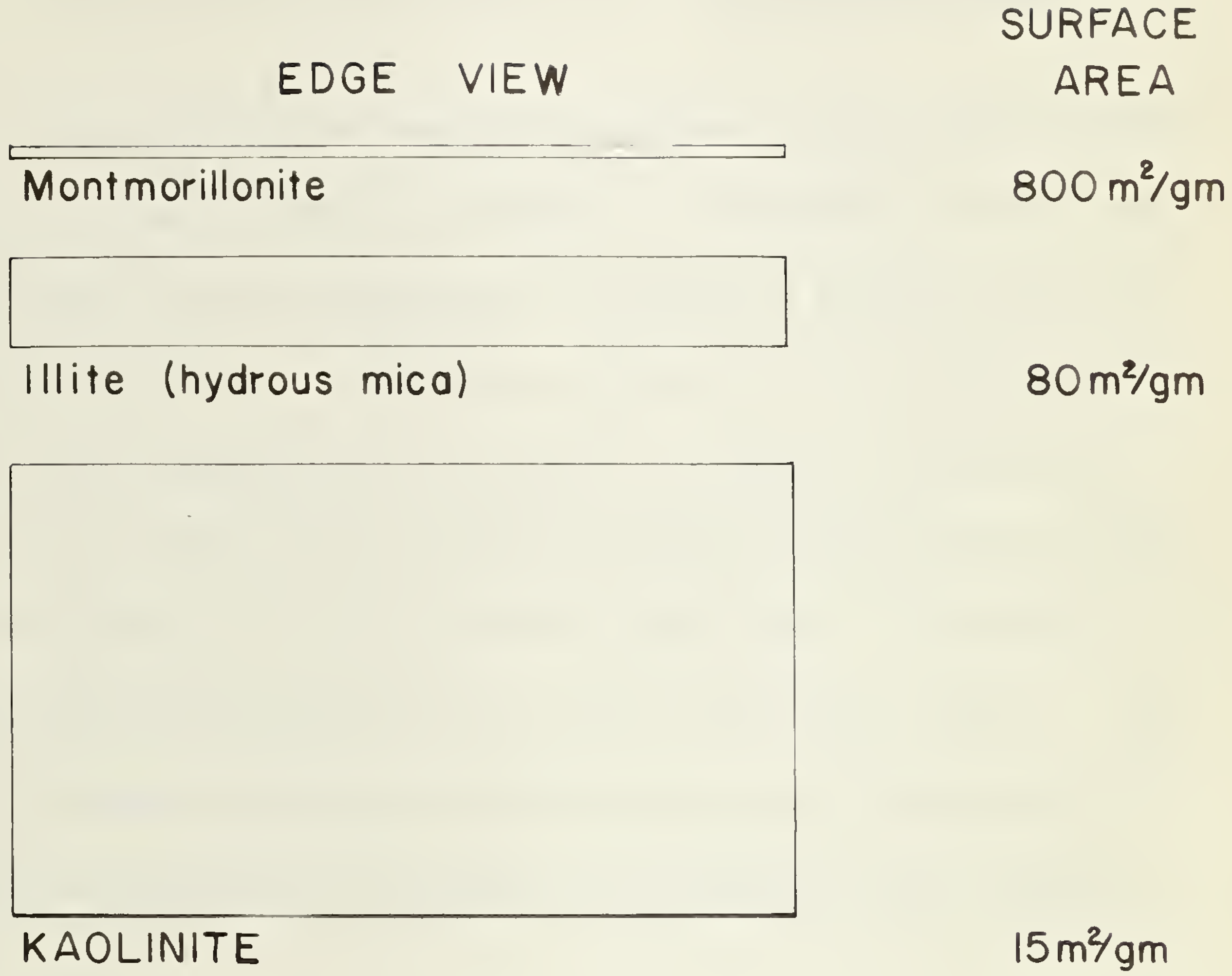

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FIGURE D1 SCHEMATIC REPRESENTION OF RELATIVE SHAPES OF CLAY PARTICLES OF THE MAIN TYPES OF CLAY (after Warkentin, 66)

are not as firmly held.

(c) The hydrogen of exposed hydroxyl groups around the edges of the crystal may be replaced by an exchangeable cation.

D.8 Grim also suggests that the importance of these causes for the three clay mineral groups is as follows. For Kaolinite broken bonds play a major role and replacement of hydrogen in hydroxyl groups is of some significance. Since there is little or no isomorphous substitution , this mechanism is of little note. It should be noted that the major charge deficiency for Kaolinite is along the edges of the crystal thus it is in these areas that adsorption takes place. The basal surfaces would be relatively clear and may be electrically neutral. The cation exchange capacity of the Kaolinite group is in the range of 3 to 15 milli equivalents per 100 grams of air dried soil. (m.e/100 gm soil).

D.9 The illite minerals also reveal their transitional characteristics in cation adsorption. Broken bonds contribute strongly but there is also some isomorphous substitution which appears to be significant. The adsorbed cations would be distributed over both edge and basal surfaces. The exchange capacity of this group ranges from 10 to 40 m.e. per 100 gm.

D.10 Isomorphous substitution within the montmorillonites appears to contribute a major share with broken bonds accounting for not more than 20% of the cation exchange capacity. The majority of the cations will thus be adsorbed on the basal surfaces of the mineral. The possibility of anion adsorption on the edges of this mineral is distinct, hence the particle may have a positive charge on the ends and a negative charge on the basal surfaces. Anion adsorption is discussed later in this appendix. There can be, therefore, the complicating factor of edge to surface attraction not present in the other minerals. The exchange capacity of this group ranges from 80 to 150 m.e. per 100 gm.

Theory of Cation Exchange

D.11 The first systematic experiments on cation exchange were carried out independently by Thompson and Way in 1850. Way in particular seemed to realize the significance of the phenomena and conducted extensive investigations. Subsequent researchers added much to the general knowledge but the mechanism of ion exchange was considered to be a simple chemical reaction. About 1890 Van Bemmelin asserted that it was a surface physical phenomena and postulated a 'hydroge' in the soil to account for ion exchange. Gedroiz in 1922 stated cation exchange was an aspect of colloidal chemistry and in turn postulated a 'zeolitic like substance' in the soil. In 1930 Hendricks and

Fry and in 1931 Kelley, Dore and Brown discovered that the inorganic colloidal material of a soil was commonly crystalline. This momentous discovery ushered in a new period of soil science and put the phenomena of cation exchange on a rational basis (33).

D.12 The kinetic theory of cation exchange cited by Baver (3) was postulated by Jenny who envisaged the mechanism as follows. "Owing to thermal energy and Brownian Movement the adsorbed cations are not at rest but oscillate and are, at times, a considerable distance from the particle wall. The cations in solution are also in motion and if one of these cations slips in behind an adsorbed cation, the pore water cation becomes adsorbed while the originally adsorbed cation passes into solution. The more loosely a cation is adsorbed the greater its amplitude of oscillation hence the greater possibility of its replacement. The converse is also true, that is a tightly held ion is more difficult to replace. The average distance of oscillation corresponds directly to the thickness of the double layer, therefore, clays having a high zeta potential contain easily exchangeable ions".

Adsorbed Water

D.13 It has been pointed out that the unbalanced charges of the clay structure are satisfied by cations adsorbed on the surface or edges of the particle. The water molecule, being a dipole, is attracted to the

clay particle causing it to hydrate. The result is a water hull around the clay particle and within this water hull are the adsorbed cations.

D.14 The dipole water molecules are oriented by the electrical force field of the soil particle, the positive hydrogens being attracted to the particle and the negative oxygens repelled. In effect, the charge of the particle is being propagated away from its surface. Low has clarified this concept to a considerable degree and the following ideas are taken from his recent paper (40). The surface of a clay mineral hydrates by the formation of hydrogen bonds between the surface hydroxyls, or oxygens, and the water molecules. The first molecular layer of adsorbed water is tightly held and succeeding layers of water are held by similar bonds to the preceding layer. As additional water layers build up the degree of bonding decreases and the order of the adsorbed water structure decreases with distance from the particle surface until free water is found. The adsorbed cations tend to break down this adsorbed water structure. Small and monovalent cations have little effect on the water structure but large or multivalent cations have a large effect. Based on this hypothesis, it appears that the sodium cation is associated with a thick adsorbed water film around the soil particle and the divalent magnesium and potassium disrupt the outer layers of adsorbed water to the point when there is only a thin adsorbed

water film on the particle. In the latter case the water molecules are strongly adsorbed thus their boundary tends to be well defined. The presence of cations and anions in the pore water would also tend to disrupt the adsorbed water structure and result in a thinning of it though the suppression of the zeta potential is also a factor since the adsorbed cations are forced closer to the clay surface.

D.15 Grim (20) concluded from a bonding strength-water content study that for a montmorillonite with sodium as the exchangeable cation the non-liquid water is three molecular layers thick (approximately 7.5A) and there is a gradual transition to oriented water molecules. Orientation may exist to 100 plus angstroms from the mineral surface. In the case of Calcium montmorillonite the non-liquid water is four molecular layers thick (approximately 10Å) and the transition to oriented water is abrupt. Orientation does not persist beyond about 15Å. The adsorbed cation has a considerable influence.

D.16 The adsorbed water surrounding the cation itself is due to the attraction of the dipole water molecule by the cation. The electric field intensity of an ion varies directly with its charge and inversely as the radius squared. It follows from this that the small ions attract the largest number of water molecules hence have the thickest water hull. Table D1 from Grim (20) lists some of the radii of the common cations.

Table D1

IONIC RADII OF SOME CATIONS

Ion	Ionic Radii, Å			
	Not Hydrated		Hydrated	
	A	B	C	D
Li	0.68	0.78	10.03	7.3
Na	0.98	0.98	7.90	5.6
K	1.33	1.33	5.32	3.8
NH_4	-	1.43	5.37	-
Mg	0.89	0.78	-	10.8
Ca	1.17	1.06	-	9.6
Ba	1.49	1.43	-	8.8
Al	0.79	0.57	-	-

A Zachariassen, W.H., Z. Krist, 80, 137 (1931)
B Goldschmidt, V., Norske. Videnskaps - Akad, Oslo 1927
C Jenny, H., J. Phys. Chem., 36 (1935)
D Pallman, H., Bodenk and Forsch., 6, 21 (1938)

Double Layer Concept

D.17 The film of water around each clay particle is probably not uniform in thickness. In Kaolinite, since most of the charge deficiency is due to broken bonds, the water hull is likely thicker on the edges. In montmorillonite the majority of the water molecules will be attracted

to the basal surfaces hence there will be a tendency for the water film to be thicker on these areas. The distribution of the adsorbed cations within this layer has posed a considerable problem and is, as yet, not fully answered. Wiklander (67) summarizes the various postulates and a brief review of this summary follows. Helmholtz first postulated a double layer as illustrated in Figure D2 (a) in which the adsorbed cations formed a monomolecular layer on the surface of the particle. There were difficulties in this theory that were brought out by the cation exchange phenomena itself and the thickness of the water film varying with the type of adsorbed cation. Gouy and Chapman independently but about the same time (1910) modified Helmholtz's postulate to a concept of a 'diffuse' double layer as illustrated in Figure D2(b). This concept pictures a decrease in the concentration of the cations as the distance from the surface of the particle increases. Its validity for low surface charge density and low concentrations appeared satisfactory but difficulties were encountered at higher concentrations. Stern presented a further modification to increase the range of validity. This modification encompasses both the Helmholtz and Gouy-Chapman concepts. It is illustrated in Figure D2 (c). It appears that increasing the concentration changes the Stern concept in the direction of the Helmholtz layer and dilution in the direction of the Gouy-Chapman concept. In the former the ion swarm shrinks and in the latter it expands.

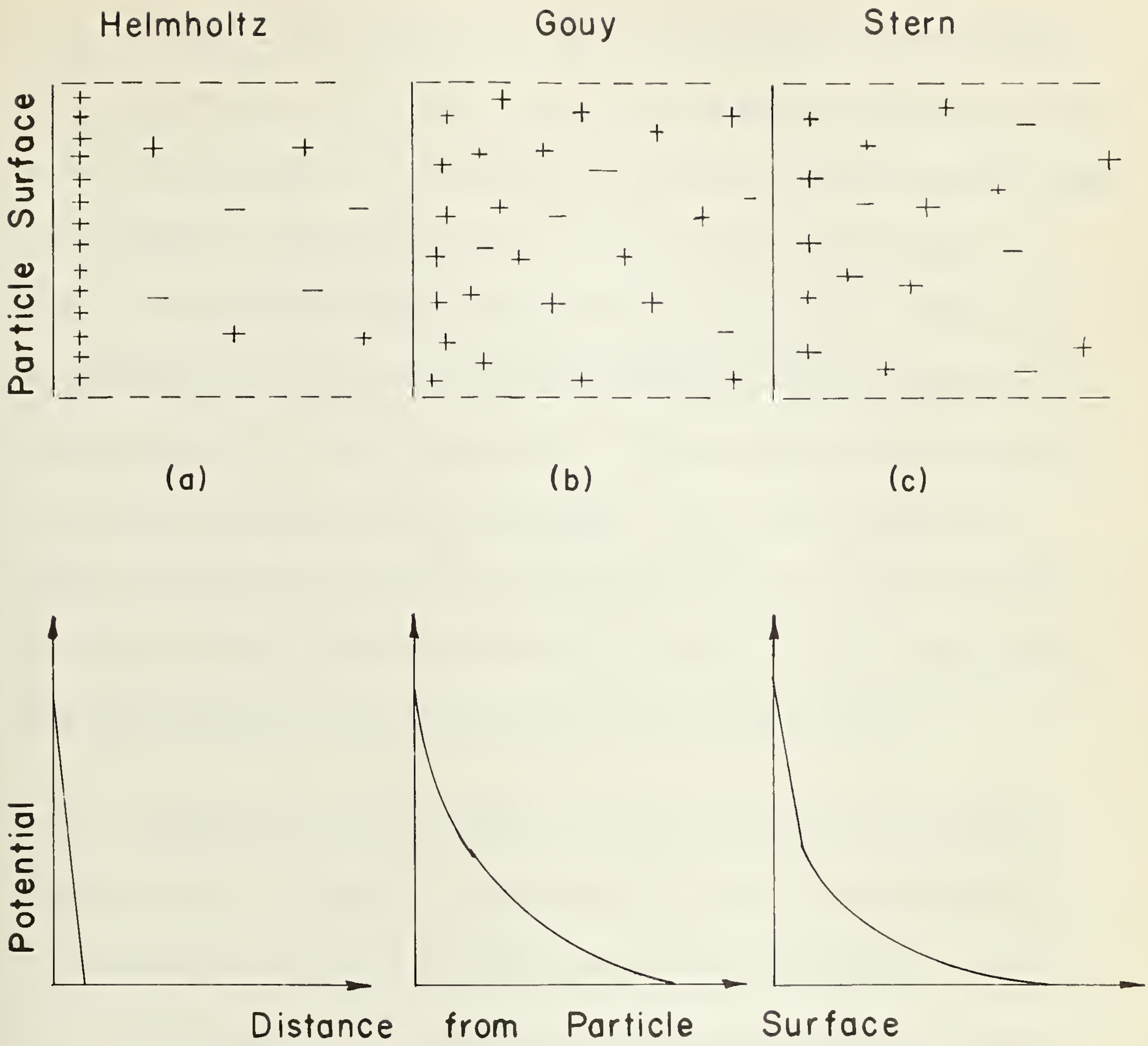


FIGURE D2 SCHEMATIC REPRESENTATION OF ION AND POTENTIAL DISTRIBUTION ACCORDING TO HELMHOLTZ, GOUY AND STERN (after Wiklander, 67)

D.18 There are in reality three layers associated with the 'double' layer concept (14). The first of these is the negative charge associated with the clay particle. The second is referred to as the immobile layer and consists of cations adsorbed so strongly to the clay surface that they do not enter into normal cation exchange reactions. It has been suggested that these cations are not hydrated and behave essentially as if they were part of the clay particle. The third layer is comprised of the mobile cations that are exchangeable. The zeta potential is a measure of charge distribution of the latter cations. The two layers that are referred to by the term 'double layer' are, first, the particle plus the immobile cations and, second, the mobile cations.

D.19 Another concept that aids in visualizing the dispersion of the adsorbed ions in a direction perpendicular to the surface to which they are attracted is presented by Winterkorn (9) and Lambe (15). Both of these sources show that the available area for adsorption of a cation is less than the hydrated radius of the cation. A monomolecular layer, therefore, cannot be formed, for example, for one type of clay investigated by Winterkorn the available area for adsorption was about 16.5 square Å and a fully hydrated sodium ion has a cross-section area of 196 square Å. The cations must be 'stacked up' away from the surface. Added to this is the thermal energy of the cation and its mutual repulsion

of other like cations. Both of these ideas lend support to the necessity of a diffusion of the ions away from the surface of the clay particle.

Zeta Potential

D.20 The clay particle and its film of adsorbed water and cations is an electrically neutral system. The net negative charge of the particle is satisfied by the cation swarm which exists over some distance from the particle surface. The electric neutrality may be considered as a system bounded by the particle surface and the outer edge of the diffuse double layer. The distance between these boundaries is a measure of a quantity termed the 'Zeta potential'. A high zeta potential appears to be associated with a thick adsorbed water film; and a low zeta potential with a thin film. Small cations appear to be associated with high zeta potentials, examples of which are lithium and sodium. Divalent magnesium and calcium have relatively low zeta potentials but it must be kept in mind that only half as many divalent cations are needed to satisfy the charge deficiency of a particle as monovalent cations. High zeta potentials (thick water hulls) lead to dispersed soil-water systems and low zeta potentials to flocculated systems. The distribution of potential as a function of the distance away from a negatively charge surface is illustrated for various

conditions in Figure D3.

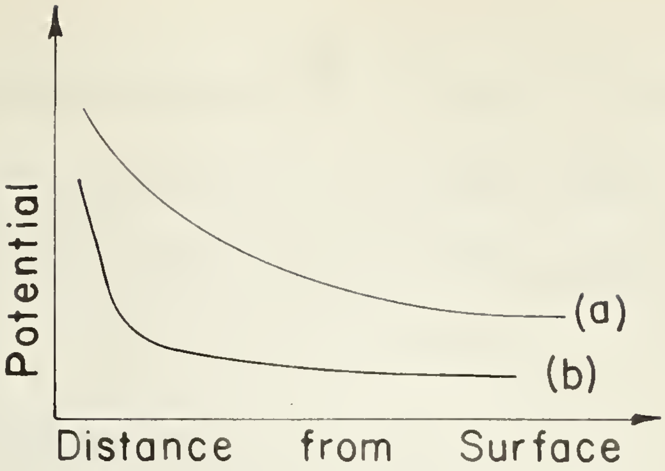
D.21 The thickness of the water film is influenced by the presence of ions in the pore water as well as by the adsorbed cation complex. Ions in the pore water cause a suppression of the zeta potential, the degree of suppression increasing as the concentration of salts in pore water increases. This suppression is probably due to physical volume occupied by the ions of the salt and to their thermal energy. The removal of the salts allows the zeta potential to assume, or tend to assume, its characteristic thickness.

Principles of Cation Exchange

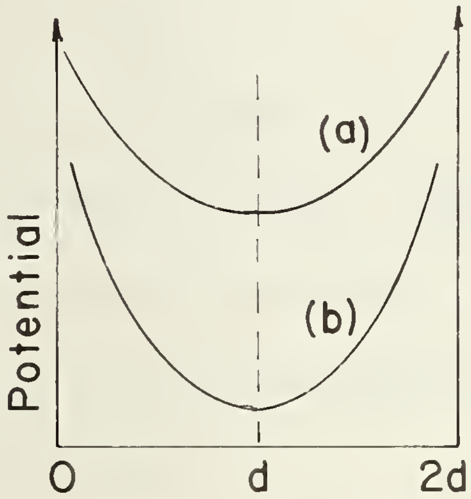
D.22 The relative replacing power of one cation for another is more complex than is inferred in previous discussion. A more complete picture may be obtained by considering the principles of cation exchange as presented by Kelley (33). Briefly these are as follows:

(a) Rate of Reaction

Experiments have shown that the exchange reaction takes place as soon as contact is established between the soil and solution. With kaolinites, vermiculites and highly hydrated montmorillonites this conclusion is not surprising since the cations are held on the surface or, as is the case with montmorillonite between the layers but inter-crystalline swelling



(i) Distribution of Potential as a function of distance from a negatively charged surface.



(ii) Distribution of potential between two negatively charged surfaces separated by a distance 2d

Curve (a) represents extensive development of the diffuse layer. Curve (b) represents conditions when a divalent cation is adsorbed or salts are present in the free water.

FIGURE D3 DISTRIBUTION OF POTENTIAL AS A FUNCTION OF DISTANCE (after Taylor, 60)

makes possible the rapid entry of cations into these spaces. For the halloysites, that occur as hollow rods, some time might elapse before the cations can diffuse into the interior of the rods.

(b) Effect of Temperature

The effects of variation of temperature are usually very small but 'optimum' temperature for the exchange reaction appears to be room temperature. It would appear, at first sight, that heating would enhance the reaction since thermal energy of the cations is involved. However, raising the temperature increases the solubility of some soil constituents and hence seriously affects the cation exchange capacity.

(c) Ratio of Soil to Solution

Usually the more dilute the suspension the greater is the exchange of the added ion for the adsorbed ion. Normally a soil-solution ratio of 10:1 is used but this is arbitrary and may be varied to suit conditions.

(d) Effect of Concentration

Usually an increase in concentration of solution added results in an increased cation adsorption but not in direct proportion, i.e. a doubling of the concentration of added salt solution does not result in a doubling of the adsorption

of the cation. The effects of concentration depend on the cation being replaced and the nature of the replacing cation. For example, a very dilute solution of sodium will result in practically no calcium replacement but on the other hand a dilute solution (.0025N) of calcium will effectively replace adsorbed sodium ions (33).

(e) Relative Replacing Power of Cations

The relative replacing power depends on the interplay of the following factors:

(i) the kind of material on which the exchangeable ion is held. For example hydrogen replaces ammonium ions only moderately from montmorillonite but with muscovite the hydrogen replaces ammonium completely.

(ii) the degree of saturation of a material with a given cation. As the amount of exchangeable calcium becomes less it is more difficult to release. On the other hand as adsorbed sodium concentration becomes less it is easier to replace.

(iii) nature of the ion. Other things being equal, the higher the valence the greater is a cation's replacing power and the more difficult it is to replace. Hydrogen is an exception since, for the most part, it behaves as a

divalent or trivalent ion. Kelley points out that the replacing power increases with atomic number for ions of the same valence.

Results of Langmuir Adsorption Theory

D.23 As may be expected there are many equations put forth to describe cation exchange each valid over a certain range for given conditions. These are fully dealt with by Kelley (33) however, only one typical equation will be presented by way of illustration and because it was used in part of the work for this thesis. This is the Langmuir adsorption theory (33, 41) which postulates, for liquid-solid interfaces, the formation of chemical bonds and when these bonds are satisfied adsorption ceases. This phenomena has been termed chemisorption. Data produced by Hamilton (26) was treated in accordance with the Langmuir theory and a straight line relationship resulted. A plot of this data is on Figure D4. These results, as well as confirming the validity of the theory for these conditions, are useful in reverse to determine the normality of a sodium acetate solution required to produce a clay having a predetermined proportion of sodium ions adsorbed on a calcium clay.

Homionic Clay

D.24 The production of ahomionic clay may be considered to follow

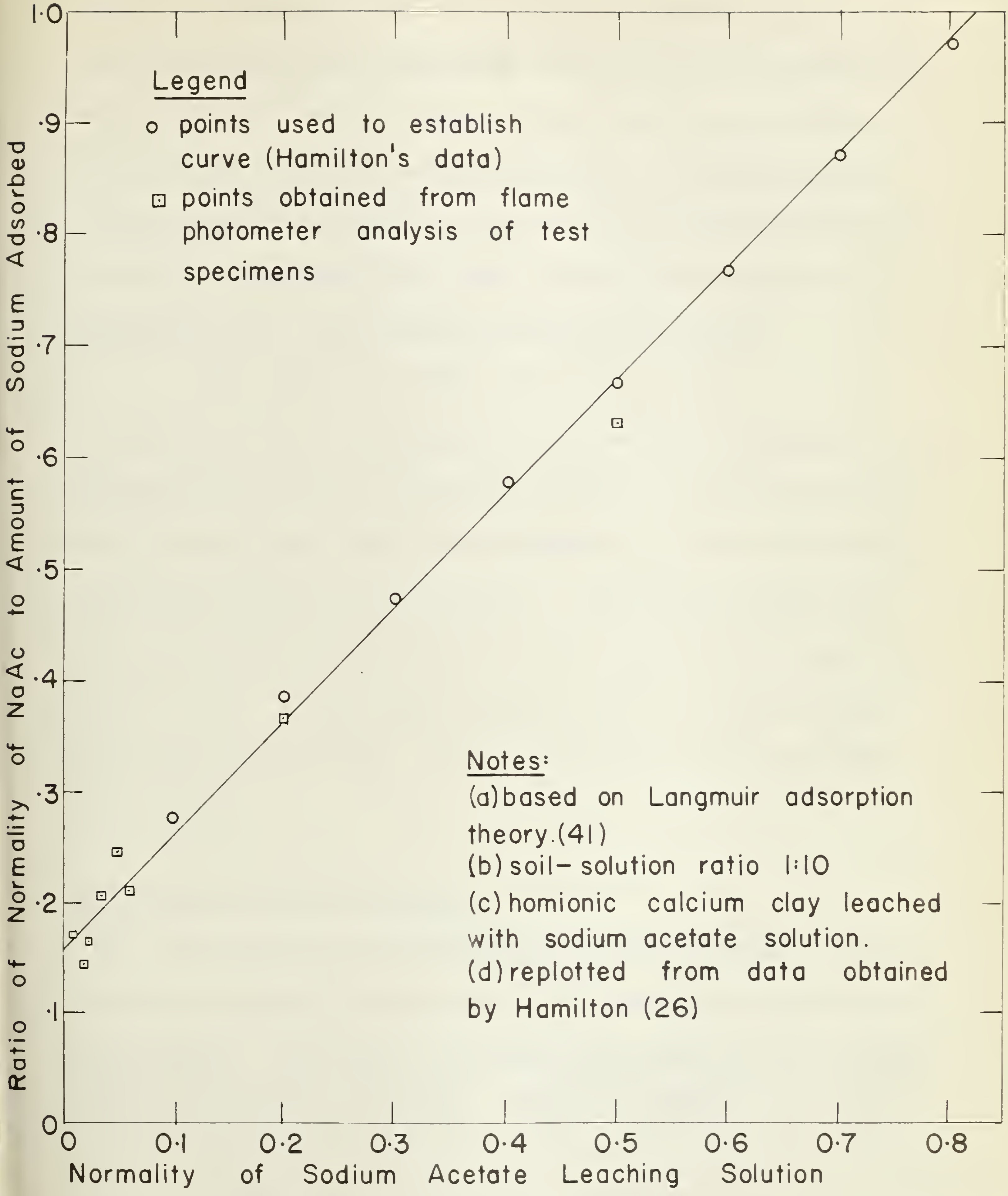


FIGURE D4 REPLACEMENT OF ADSORBED CALCIUM CATIONS BY SODIUM CATIONS

from the preceding discussion. Inherent in the Langmuir theory is the fact that an equilibrium is established between the adsorbed cations and those in the added solution. This, in turn, infers the applicability of the Law of Mass Action to exchange phenomena. According to Davis (11) this inference is not strictly true but may be considered sufficiently precise for the purposes of this thesis and mass action equations will be presumed to hold. If the following equation is considered:

$$(\text{Ca - clay}) + \text{Na}^+ \rightleftharpoons (\text{Na - clay}) + \text{Ca}^{++}$$

the equilibrium that is established can be forced toward the right, i.e. to the production of sodium clay, by removal of the calcium ions appearing on the extreme right. This is accomplished by removal of the solution and renewal with fresh sodium solution. About four applications of a one normal acetate solution of the salt of the desired cation results in a homionic state. Final washing with ethyl alcohol removes the excess cations from the pore water. This is the procedure used in the work of this thesis.

Measurement of Adsorbed Cations

D.25 The quantitative determination of the cation exchange capacity of a soil is a relatively simple procedure but demands a careful laboratory technique. The details of the determination may be found in Appendix II of P. Thomson's thesis (62). Briefly the technique is to replace

the adsorbed cations on a soil by ammonium ions by leaching with ammonium acetate. The leachate from this step contains the original adsorbed cations and any cations that were present in the pore water. It is retained for flame photometric determinations of the individual species present. The original sample is then washed with alcohol to remove any ions from the pore fluids. This step ensures that all the ammonia present is adsorbed by the soil. These ammonia ions are in turn replaced by leaching with a sodium chloride solution hence the leachate from this step contains all the adsorbed ammonia ions. A nitrogen determination on this leachate provides a quantitative measure of the amount of ammonia hence the exchange capacity of the soil.

D.26 The principle of the flame photometer takes advantage of the fact that each ion when vapourized imparts a characteristic colour to the flame. Machine settings isolate this particular wavelength of light and measure the intensity of emission. The latter is a function of the amount of the ion present. The machine readings for an unknown are compared to a standard curve established using solutions of known strength. Interpolation allows the quantitative determination of the amount of the given ion present. Each ion must be analysed separately.

D.27 It will be noted that the cation exchange capacity as determined by the ammonia procedure is that of the soil itself. The total of the

flame results, found by summing the results for each ion, contains any cations that were present in the pore water, hence this total is usually the larger of the two methods. The amount by which the latter is in excess of the former is a measure of the soluble salts in the pore water. The pore water cations may also be determined separately on a sample of pore water obtained from the soil by aspiration.

D.28 The most common cations found on the soil in the field are calcium, magnesium, potassium, sodium and hydrogen. The four metallic cations normally occur in abundance in the given order. In the soils of Alberta usually calcium and magnesium predominate occupying up to 95% of the exchange positions. However, in the top few feet of some soils, sodium has accumulated and occupies up to 25% of the exchange positions.

Anion Exchange

D.29 There is one interesting but complicating factor arising out of the consideration of the distribution of the charge and hence the configuration of the water hull on a clay particle. It will be recalled that the seat of the charge of a montmorillonite particle is the octahedral layer thus the thickest part of the water hull exists on the basal surface. Broken bonds occur on the edges of the crystal and an OH group may attract anions which, in turn, will impart a negative charge to the end

[illegible]

[illegible]

[illegible]

[illegible]

of the particle (64). The net overall particle charge will still be negative. The complicating factor is the end-to-surface attraction possible between particles.

D.30 The field of anion exchange has not been as intensively investigated as has cation exchange though some work has been done (53 and 54). Grim (20) suggests that most of the anions are adsorbed around the edges of the mineral. Thus for kaolinite both the cation and anion exchange should be approximately equal and such is the case. For montmorillonite since broken bonds are a small portion of the cation exchange capacity, anion exchange may be expected to be a small proportion of the cation exchange capacity. There are many complications that may arise in measuring anion exchange such as chemical reaction with alumina which results in destruction of the clay micelle and combination of the anion with any free iron, aluminum or alkaline earths thus the anions are not adsorbed but enter into a reaction. The common anions are phosphate, sulphate, nitrate and chlorine. In the work of this thesis the large acetate anion was used which, as well as not being adsorbed, has the further advantage of being soluble in alcohol thus facilitating its removal by washing. As anion exchange does not directly affect this work, it will be left with just this brief reference.

Summary

D.31 A great deal of material has been touched upon in this appendix in an attempt to prepare a background and to provide an introduction to the part that soil physics can play in an understanding of some of the phenomena of soil mechanics. Since surface forces are dependent on the surface area available this topic was discussed. It was shown that particle size, shape, and composition are all important. The causes of cation exchange were given as broken bonds, isomorphous substitution and the hydrogen of exposed hydroxyls being replaced by an exchangeable cation. The water hull surrounding a clay particle was presented and the postulates dealing with the distribution of cations within this water film were considered. These postulates are the Helmholtz double layer, the Gouy-Chapman diffuse double layer and the Stern modification. The distance over which the adsorbed cations are diffused from the particle wall is a measure of the zeta potential. The mechanism of the exchange of one cation for another as proposed by Jenny was briefly given and the principles of cation exchange were outlined. These principles consider the rate of reaction, affect of temperature, the ratio of soil to solution, concentration of added solution and the relative replacing power of cations. The production of homionic clay takes advantage of the general applicability of the Law of Mass Action by driving the equilibrium in the desired direction. The

measurement of the cation exchange capacity is a simple double leaching procedure. The first fills all the exchange positions with ammonium ions and the second leaching removes them. The determination of the amount of ammonium removed leads to the cation exchange capacity. The flame photometer is used to determine quantitatively the amount of each type of ion present. A brief consideration of anion exchange rounded out this appendix.

APPENDIX E

ORIGIN OF CLAY MINERALS

APPENDIX E

ORIGIN OF CLAY MINERALS

Introduction

E.1 The soils engineer must concern himself with the question of how clay minerals originate if he is to view, in its proper perspective, the part played by clay minerals in Soil Mechanics. The knowledge held by the soils engineer need be neither comprehensive nor exhaustive but should be general in order that he may avail himself of the expert knowledge of the clay mineralogist or the soil scientist. It is the purpose of this appendix to present general considerations of the origin of clays. In order to achieve this purpose it is necessary to review, in the briefest manner possible, the agricultural classifications and the work of the soil scientist.

E.2 Clay minerals are a secondary product of the weathering or alteration of rock forming minerals (3, 23, 72)*. Their formation may be considered to occur in-situ then to undergo transportation by the natural agencies of the geologic cycle. The glaciers undoubtedly

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* Numbers in parentheses refer to references in the Bibliography of this thesis.

contributed in a large measure to the distribution of clay minerals in Alberta. The in-filling of the pre-glacial valleys leading to the existence of clays at considerable depths is an example of the work of the continental glaciation. Burial of paleosols by glacial drift may also account for clay minerals existing at depths much in excess of the depth of weathering. Forman and Brydon (13) discuss the clay mineralogy of Canadian Soils and the distribution of clay minerals in Canada. In very general terms, they indicate montmorillonite to be common in Western Canada and illite in the east. Other interesting and serious problems, some of which involve the clay minerals, are outlined by Legget and Hardy in a recent paper (38).

E.3 The engineer, for the most part, is not directly concerned with the topsoil other than taking care to remove it. Here, the term topsoil is meant to include the upper layer of the solum that contains high percentages of organic material. There are, however, occasions when the soil of the agronomist poses problems to the engineer. These problems can have an economic aspect, for example the use of good farmland for airfields or to be flooded by reservoirs. They may also include technical aspects such as the depth to which stripping should be carried before road construction. It would appear that the latter problem should be related to the soil forming processes that have or are

taking place. Much information on the upper few feet of the soil is available in the form of maps issued by Federal or Provincial Departments of Agriculture. The Department of Highways of Ontario has been using these maps extensively since 1945 and at present are undertaking studies to correlate geotechnical properties of the soil with the characteristics mapped by agricultural personnel (55).

Soil Science of the Agronomist

E.4 Soils are classified for agricultural purposes on the basis of morphological or physical properties such as macroscopic structure or texture and its grade, class and colour (72,36). The soil profile is considered to consist of 3 horizons, A, B and C. The A horizon is the uppermost and is characterized by being the zone of maximum removal of material by leaching and the zone of maximum accumulation of organic material. The B horizon, which underlays the A horizon, is characterized by the accumulation of materials removed from the A horizon, a concentration of in-situ weathering products, which are usually the clay minerals, an oxidation of the sesquioxides that impart to it a different colour from the A horizon and a different structure from the horizons either above or below. The C, or lowest horizon, is the original soil relatively unaffected by the pedogenic processes (36). Somewhat anologous to the Unified Soil Classification system, the agricultural

worker has a letter code used in conjunction with the horizon characters, A, B, and C. To illustrate, a lower case k designates a horizon enriched with carbonates; c a horizon from which the clays have been removed; etc. The depth of soil development, or of leaching, usually extends to a depth of only a few feet in Canada. Leahey (36) suggests the lower limit usually considered for classification purposes to be about four feet.

E.5 The soil scientist is interested in the soil weathering processes that bring about the changes in a parent material that result in a medium capable of supporting plant growth. This conversion affects the soils engineer by modifying the material with which he is going to deal and by being the processes involved in the production of the clay minerals. Typical processes are given with brief explanatory comments:

(a) Oxidation

$$4\ FeO + O_2 \rightarrow 2Fe_2O_3$$

(b) Hydration - the conversion of hematite to limonite

$$2Fe_2O_3 + 3H_2O \rightarrow Fe_2O_3 \cdot 3H_2O$$

(c) Hydrolysis - the first step in the conversion of orthoclase to kaolinite

$$KAlSi_3O_8 + H_2O \rightarrow HAlSi_3O_8 + KOH$$

(d) Formation of Soluble salts which may be removed to the

water table and subsequently to rivers.

$$2KOH + CO_2 \longrightarrow K_2CO_3 + H_2O$$

(e) Formation of Kaolinite

$$2HAlSi_3O_8 + H_2O \qquad H_4Al_2Si_2O_9 + 4SiO_2$$

The first term on the right is kaolinite and the second term is amorphous silicate which may take the form of opal, chalcedony etc.

The last three of these equations can be written as one equation showing the conversion of orthoclase to kaolinite, viz.,

$$2KAlSi_3O_8 + 2H_2O + CO_2 \longrightarrow K_2CO_3 + H_4Al_2Si_2O_9 + 4SiO_2$$

The Plagioclase series weathers in a similar manner.

E. 6 The clay minerals themselves may undergo weathering according to the following scheme:

Micas $\rightleftharpoons$ Illite $\rightleftharpoons$ Intermediate Minerals $\rightleftharpoons$ Chlorite $\rightleftharpoons$ Vermiculite $\rightleftharpoons$ Montmorillonite $\rightleftharpoons$ Kaolinite $\rightleftharpoons$ Gibbsite

This is a reversible reaction depending on the environment which includes parent material and the ions in solution. One example is the leaching of potassium from illite to form expanding lattice minerals. Montmorillonites may form directly from the weathering of chlorites or vermiculites and also as a result of complex processes from other clay minerals such as the micas.

Sedimentation

E.7 There appears to be little change in the physical characteristics as weathered material is stripped from the land surface, transported and deposited in fresh water (23). However, deposition into salt water can bring about substantial changes in argillaceous materials the most common of which appears to be the uptake of potassium and the formation of illites. Large amounts of magnesium favour the formation of attapulgites and high sodium contents favour zeolites.

Metamorphism

E.8 Grim (20, 21, 23) states that the application of tectonic forces squeezes out the water from between the sheets and tends to transform the montmorillonites in the direction of the illites, to clay mineral micas thence to muscovite, biotite, etc. Intense metamorphism may complete the transformation to non-hydrous crystalline compounds. The preconsolidated clays of Western Canada may fit into this scheme being in the very early stages of the metamorphism.

Bentonites

E.9 Bentonite is not a clay mineral but is a clay that is considered to form from the in-situ weathering of volcanic ash which appears to have been deposited in bodies of water (21, 23). It contains the clay mineral montmorillonite and thus posesses the basal cleavage, high

swelling and low strength characteristics of this group. It may form in thin layers which may distate the stability of a slope and yet the layers may be difficult to detect by normal sampling techniques.

Salts in the Pore Water

E.10 The presence of salts in the pore water of a soil mass assume a position of some importance during considerations of strength of the soil. These salts originate in two general ways, being dissolved from rocks or as a by-product of soil weathering. An example of the latter procedure is given in Paragraph E.5 (d). These salts are carried to rivers either on or below the surface by moving water. It has been suggested that many of the glacial lakes may have had an appreciable salt content. These salts remain in the pore water during deposition and during subsequent uplift or drainage of the lakes, evaporation of the water leaves the salts behind to become more concentrated and eventually to precipitate out. It is entirely conceivable that during consolidation water will migrate out and leave the salts behind. This latter effect could be explained on the basis that interlocked water films act as a membrane permeable to water but not to ions. Another hypothesis is a sort of shunting action of one water molecule on another until some molecule near the surface is squeezed out of the mass but the salt molecules remain behind.

Summary

E.11 The clay minerals are a secondary product of the weathering or alteration of the rock forming minerals. They have been redistributed in Alberta by the glaciers as well as continuously forming in place under the influence of the soil forming processes. Typical processes were outlined for the formation of kaolinite and a scheme suggested for the formation of other clay minerals. A study of soil formation is a broad and complex field and only a brief outline can be given. As well as being a product of soil weathering, sedimentation and metamorphism play a part in clay mineralogy. The aim of this appendix was to present a brief outline of the origin of the clay minerals which, it is felt, are a concern of the soils engineer.

APPENDIX F

DATA SHEETS FOR TESTS ON NATURAL SOIL

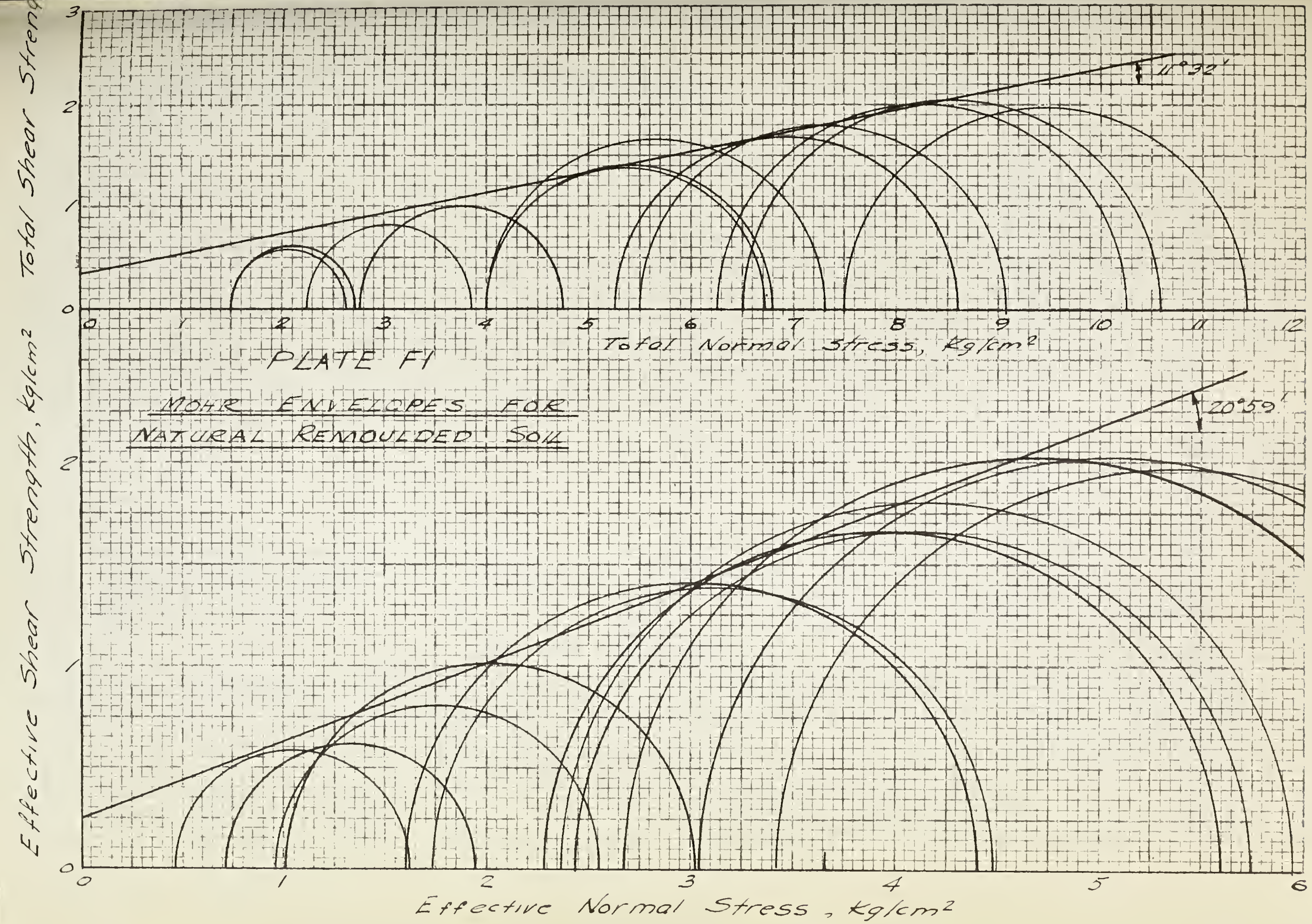
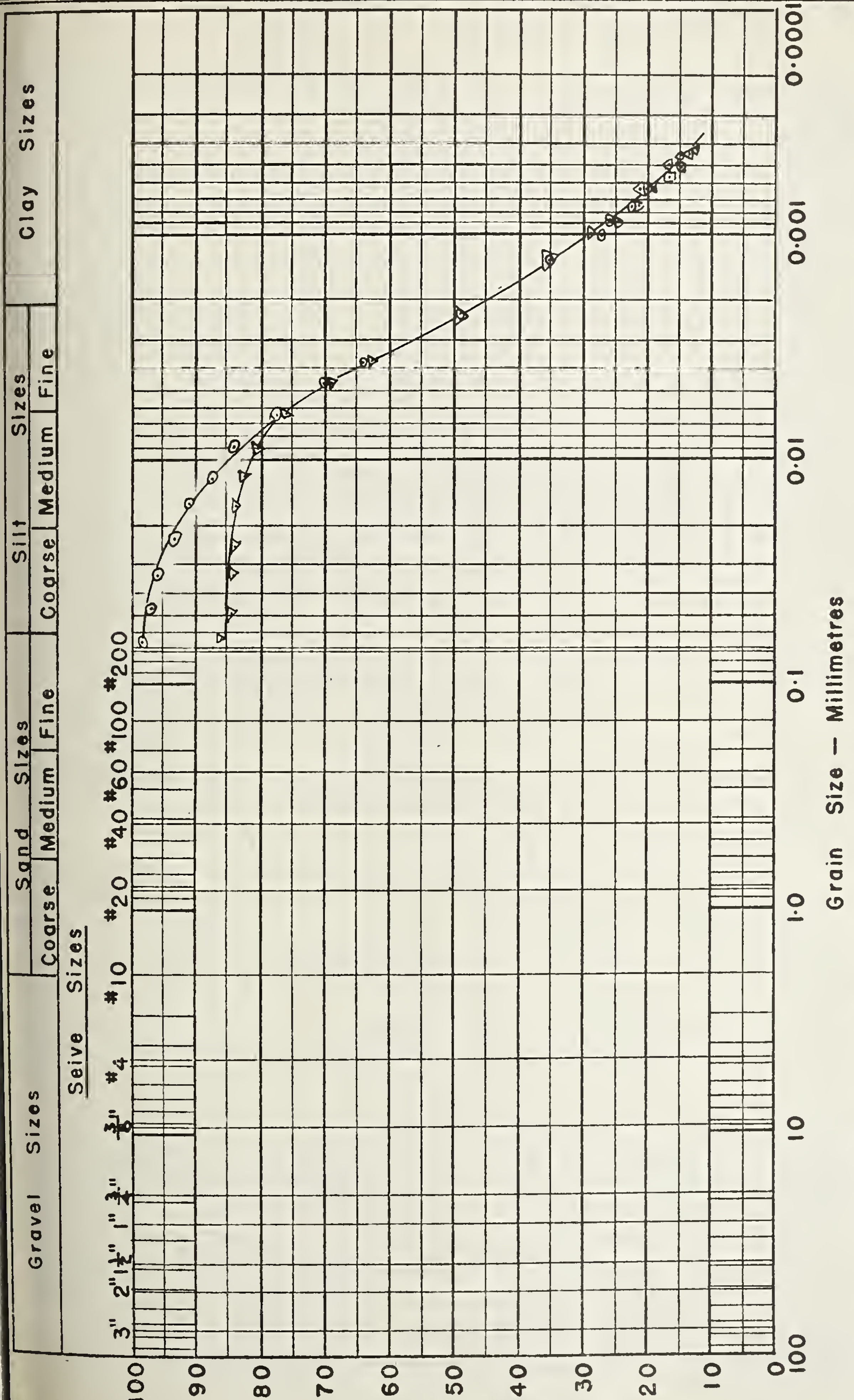


PLATE F1

MOHR ENVELOPES FOR NATURAL REMOULDED SOIL

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY
GRAIN SIZE CURVE

PROJECT *THESIS*
SITE
SAMPLE *Natural*
LOCATION
HOLE DEPTH
TECHNICIAN *ST & DW* DATE *19 May 61*



D₁₀ = ____ mm.
D₆₀ = ____ mm.
Cu ____

Remarks: Sample No 1 - ⊙ ; No 2 - Δ

From Wk of Hamilton: $W_L = 76.1$

$W_P = 28.9$

$I_P = 47.2$

Activity = $\frac{I_P}{\%\ clay} = \frac{47.2}{45} = 1.05$

Note: M·I·T· Grain Size Scale

PLATE F2

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

HYDROMETER TEST

PROJECT THESIS
SITE
SAMPLE Natural #1
LOCATION
HOLE DEPTH
TECHNICIAN DW DATE 12 May 61

ate	Temp. °C	Time	Elapsed Time	R_h'	R_h $=R_h'+c_m$	D m.m.	$R_h+m_t-c_d$	W.%	W % Basis Orig. Sample	Remarks
May61		1345	~~1345~~	0						
			.5	22.3	22.8	.067	22.9	98.9		
			1m	22.0	22.5	.048	22.6	97.6		
			2	21.7	22.2	.034	22.3	96.3		
			4	21.0	21.5	.024	21.6	93.3		
	22.0		8	20.5	21.0	.017	21.1	91.1		
			15	19.7	20.2	.013	20.3	87.6		
	21.9		30	18.9	19.4	.0091	19.5	84.2		
	21.7	1445	1hr	17.5	18.0	.0065	18.0	77.7		
	21.4	1545	2	15.7	16.2	.0047	16.2	70.0		
	21.2	1645	3	14.4	14.9	.0039	14.8	63.9		
	20.6	2249	9:04	11.0	11.5	.0023	11.3	48.8		
3 May	19.9	1710	27:~~4~~25	7.9	8.4	.0014	8.1	35.0		
4✓	19.7	2105	55:20	6.1 4.7	6.6 ~~4~~5.2	.001 .00063	6.3 4.9	27.2 21.2		
5✓	19.7	0905	67:20	5.8 4.3	6.3 4.8	.0009 .00062	5.9 4.4	25.5 19.0		
✓	20.5	1050	93:05	4.8 3.5	5.3 4.0	.00075 .0005	5.1 3.8	22.0 16.4		
✓	20.6	0845	115:00	3.2	3.7	.00045	3.0	13.0		
✓	20.8	1645	123.00	3.1	3.6	.00043	2.9	12.5		

ydrometer No.s 9857R and 50700R Graduate No.

$W\% = \frac{100}{W_s}\cdot\frac{S_s}{S_s-1}(R_h+m_t-c_d) =$ 4.32 $(R_h+m_t-c_d)$

eniscus correction = c_m = 0.5 and 0.5 respectively

ispersing agent used Colgon Amount 10ml

orrection for change in density of liquid due to addition of dispersing agent = c_d

c_d = 0.3 and 0.3 respectively

pecific Gravity of Solids = G_s 2.80

escription of Sample
natural soil crushed to
pass N° 40 sieve - air dried

hygroscopic m.c. = 3.89%

Method of Preparation
100ml 6% H_2O_2 - 1hr at 105°C
718hrs to cool - covered

Remarks

Initial Moisture Content

ontainer No.
t. Sample Wet + Tare
t. Sample Dry + Tare
t. Water
are Container
t. of Dry Soil
nitial Moisture w %

Dry Weight of Sample

Container No.
Wt. Sample (~~Wet~~/Dry) + Tare 775.79
Tare 739.77
Wt. (~~Wet~~/Dry) Soil 36.02
Dry Weight from Initial
Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist.}\%}$ =

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SOIL MECHANICS LABORATORY

HYDROMETER TEST

PROJECT *THESIS*
SITE
SAMPLE *Natural Clay - #2*
LOCATION·
HOLE DEPTH
TECHNICIAN *DW* DATE *12 May 61*

ate	Temp· °C	Time	Elapsed Time	R'_h	R_h $=R'_h+c_m$	D m.m.	$R_h+m_t-c_d$	W·%	W % Basis Orig ·Sample	Remarks
May61		1403	0							
			.5m	19.6	20.1	.068	20.2	85.8		
			1	19.4	19.9	.049	20.0	85.0		
			2	19.2	19.7	·033	19.8	84.2		
			4	19.1	19.6	·025	19.7	83.7		
	22.0		8	19.0	19.5	·017	19.6	83.3		
			15	18.9	19.4	·013	19.5	82.9		
	21.8		30m	18.5	19.0	·0092	19.0	80.7		
	21.6	1503	1hr	17.3	17.8	.0065	17.8	75.6		
	21.4	1603	2hr	15.8	16.3	.0047	16.3	69.3		
	21·2	1658	2:55	14.3	14.8	.0039	14.7	62.5		
	20.6	2245	8:42	11.0	11.5	.0024	11.3	48.0		
May61	19.9	1710	27:07	8.0	8.5	.0014	8.2	34.9		
〃 〃	19.7	2110	55:07	6.3 4.4	6.8 4.9	.001 .0006	6.5 4.6	27.6 19.6		
〃 〃	19.7	0913	67:10	5.6 3.7	6.1 4.2	.0009 .00057	5.8 3.9	24.6 16.6		
〃 〃	20.5	1055	92:52	4.7 3.2	5.2 3.7	.00075 .0005	5.0 3.5	21.2 14.9		
〃 〃	20.6	0850	114:47	3.1	3.6	.00045	3.4	14.5		

ydrometer No.s *9857 R* and *50700 R* Graduate No.

$q_0=\frac{100}{W_s}\cdot\frac{S_s}{S_s-1}(R_h+m_t-c_d)$ = *4.25* $(R_h+m_t-c_d)$

eniscus correction = c_m = *0.5* and *0.5* respectively

spersing agent used *Calgon* Amount *10 ml*

orrection for change in density of liquid due to addition of dispersing agent = c_d

c_d = *0.3* and *0.3* respectively

pecific Gravity of Solids = G_s *2.80*

escription of Sample

Method of Preparation

Remarks

Initial Moisture Content

ontainer No.
t. Sample Wet + Tare
· Sample Dry + Tare
· Water
re Container
· of Dry Soil
tial Moisture *w* %

Dry Weight of Sample

Container No.
Wt. Sample (~~Wet~~/Dry) + Tare *766.35*
Tare *(DK)* *729.69*
Wt· (~~Wet~~/Dry) Soil *36.66*
Dry Weight from Initial
Moisture= $\frac{100 \times \text{Wt· Wet Soil}}{100+\text{Init· Moist.}\%}$ =

UNIVERSITY of ALBERTA DEP'T. of CIVIL ENGINEERING SOIL MECHANICS LABORATORY **SPECIFIC GRAVITY**	PROJECT THESIS SITE SAMPLE Natural LOCATION HOLE DEPTH TECHNICIAN DATE 10 MAY 61

ple No. 1 TRIAL No	1	2	3	4
k No.	109	101	109	101
hod of Air Removal	Vacuum	Vacuum	Vacuum	Vacuum
w+s	701.28	692.59	704.29	689.68
perature T	23.1°C	23.25	24.7°C	28.4°
-w	671.79	667.55	671.56	667.11
porating Dish No.				
Sample Dry + Dish				
e Dish				
	45.82	39.30	51.2	35.92
	2.80	2.76	2.80	2.69(?)

W_{b+w+s} = Weight of flask + water + sample at T°.

W_{b+w} = Weight of flask + water at T° (flask calibration curve).

W_s = Weight of dry soil

G_s = Specific gravity of soil particles $= \frac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

ermination of W_s from wet soil sample:

ple No.	1		Sample No.		
tainer No.	V-31		Container No.		
Sample Wet + Tare	90.7962		Wt. Test Sample Wet + Tare		
Sample Dry + Tare	89.6493		Tare Container		
Water	1.1469		Wt. Test Sample Wet		
e Container	60.1970		W_s		
of Dry Soil	24.4523				
sture Content w %	3.89				

scription of Sample: Natural Soil, Pass N° 40 Sieve

Trial	1	2	3	4
Tare + Soil	221.30	210.02	226.71	206.48
Tare	173.~~52~~ 70	169.16	173.52	169.16
Airdry Wt Soil	47.60	40.86	53.19	37.32
marks: Wt Soil	45.82	39.30	51.20	35.92

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Department of Civil Engineering
Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Natural
Depth sample 1
Engineer ST
Technician
Date of Set-up 11 Jan 61
Test Lateral Pressure 1.5 kg/cm^2

e Description Average Cation
alysis: Exchange Capacity 28.2 me
100gm soil; Ca 84.5%; Na 0.5%,
0.8%, Mg 14.2%; Salts 73.6 me./100gm soil
5 internal wicks
Weight 135.371 gms

mm 1. 80.06 2. 80.08 Aver: 80.07 Area Top A_T 9.809 sq.mm
m Top 1. 35.32 2. 35.36 Aver: 35.34 Area Centre A_C 10.072 sq.mm
entre 1. 35.82 2. 35.80 Aver: 35.81 Area Bottom A_B 10.145 sq.mm
ottom 1. 35.92 2. 35.94 Aver: 35.93 Average X-Sect Area
Volume 80.27 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 10.02\ \text{cm}^2$

IDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
230	0	22.70	
	0.1	21.72	
	0.25	21.65	
	0.5	21.55	
	1.0	21.35	
	1.5	21.25	
	3	20.87	
	4	20.6	
	6	20.3	
	8	19.97	
	10	19.65	
	15	18.97	
	20	18.37	
	30	17.45	
	40	16.75	
	60	15.70	
	115	14.10	
	170	13.35	
	230	12.90	
	300	12.57	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
12 Jan 2140	550	11.90	
13 " 0900	1230	11.03	
" " 1215	1425	10.85	
" " 1745	1755	10.52	
14 Jan 0930	2700	9.95	
15 " 1430	4440	9.10	
16 " 0945	5595	9.20	
17 " 1730	7500	9.00	
19 " 1130	10,020	8.95	
22 " 1315	14,445	8.85	
25 " 1715	19,005	8.82	
27 " 1730	21,900	8.82	
30 " 0930	25,740	8.82	13.88

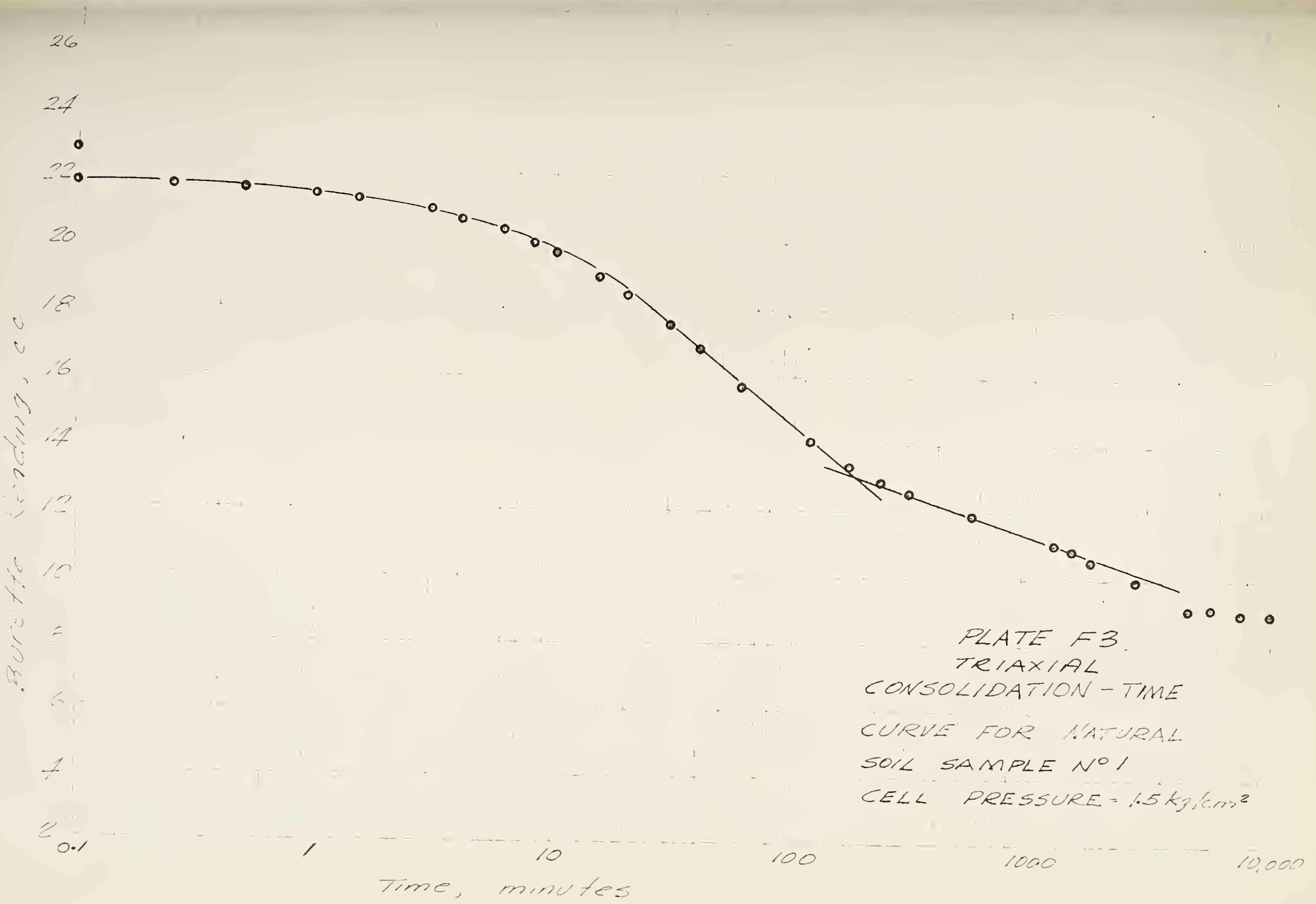
PLATE F3.
TRIAXIAL
CONSOLIDATION - TIME
CURVE FOR NATURAL
SOIL SAMPLE N° 1
CELL PRESSURE = 1.5 kg/cm²
Burette Reading, cc
26
24
22
20
18
16
14
12
10
8
6
4
2
0.1
1
10
100
1000
10,000
Time, minutes

PLATE F3

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Soil Mechanics Laboratory

COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Natural
Depth Sample 1
Engineer ST
Technician
Date of Test 30 Jan 61
Test Lateral Pressure σ_3 = 1.5 kg/cm²
Back Pressure 2.0 kg/cm²
Remarks
Rate of Strain 600 min per 10 mm
Proving Ring № 320-0125
Area Correction Factor 1.14

ailure
3) = 1.20 σ_1 = 2.70
0.77
1.93
.77
6.0 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_p}{\sigma_1 - \sigma_3}$ %
0	10.02	0	0	0	-.10					
2	10.02	15	.098	0.17	-.04	1.71	1.54	1.11	.025	
5	10.03	25	.0969	0.28	-.02	1.80	1.52	1.18	.06	
20	10.04	62	.0939	0.66	+.18	1.98	1.32 ~~1.98~~	1.50	0.25	27.2
40	10.07	80	.0931	0.84	+.34	2.00	1.16	1.72	0.50	40.5
60	10.10	88	.0928	0.92	+.43	1.99	1.07	1.86	0.75	46.7
80	10.12	94	.0927	0.98	+.47	2.01	1.03	1.95	1.00	48.0
1.10	10.16	100	.0925	1.04	+.52	2.02	0.98	2.06	1.375	50.0
120	10.17	101	.0925	1.05	+.52	2.03	0.98	2.07	1.50	49.5
140	10.20	103	.0924	1.06	+.55	2.01	0.95	2.12	1.75	51.8
160	10.22	106	.0924	1.09	.59	2.00	0.91	2.20	2.00	54.1
180	10.25	108	.0923	1.11	.64	1.97	0.86	2.29	2.25	57.6
200	10.28	109	.0923	1.12	.65	1.96	0.85	2.31	2.50	58.0
220	10.30	111	.0923	1.13	.66	1.97	0.84	2.35	2.75	58.4
240	10.33	112	.0923	1.14	.68	1.96	.82	2.39	3.00	59.5
260	10.35	114	.0922	1.16	.69	1.97	.81	2.43	3.25	59.5
280	10.38	115	.0922	1.16	+.70	1.96	.80	2.45	3.50	60.3
300	10.41	116	.0922	1.17	.72	1.95	.78	2.50	3.75	61.5
320	10.43	117	.0922	1.18	.73	1.95	.77	2.53	4.00	61.9
340	10.46	117.5	✓	1.18	.74	1.94	.76	2.55	4.25	62.6
360	10.49	119	✓	1.19	.75	1.94	.75	2.59	4.50	63.0
380	10.52	119	✓	1.19	.76	1.93	.74	2.61	4.75	63.9
406	10.55	120.5	.0921	1.20	.77	1.93	.73	2.64	5.07	64.1
420	10.57	121	✓	1.20	.77	1.93	.73	2.64	5.25	✓
440	10.60	121	✓	1.20	.77	1.93	.73	2.64	5.50	✓
460	10.63	121.5	✓	1.20	.77	1.93	.73	2.64	5.75	✓
480	10.66	122	✓	1.20	.77	1.93	.73	2.64	6.00	✓
500	10.69	123	✓	1.21	.78	1.93	.73	2.64	6.25	✓
520	10.71	123	✓	1.21	.78	1.93	.73	2.64	6.50	✓

Sheet 2

F9

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Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth

Sample 1

Area Correction Factor 1.14

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma 1 - \sigma 3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1 - \sigma_3}$
700	10.98	131	.0920	1.25★	.77	1.98	.73	2.71	8.75	61.5

★specimen has a marked "S" shape - Loading Piston rubbing against cap - specimen considered failed

ontent - Initial 52.2 %

Final 42.27

aturation - Initial 96.2

Final

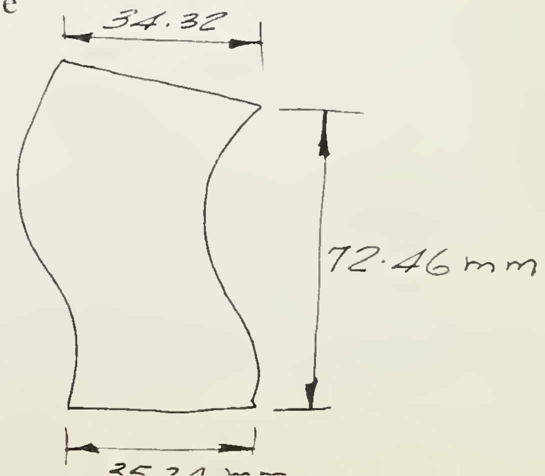
can not obtain final Vol. %c specimen too soft

Ra - Initial 1.51

Final

ure Reaction 100%

Sketch of Failure



THE HYDRO-ELECTRIC POWER COMMISSION OF ONTARIO
PRINTING DEPT
H.E.P.C. NO. 403 5M-5-48

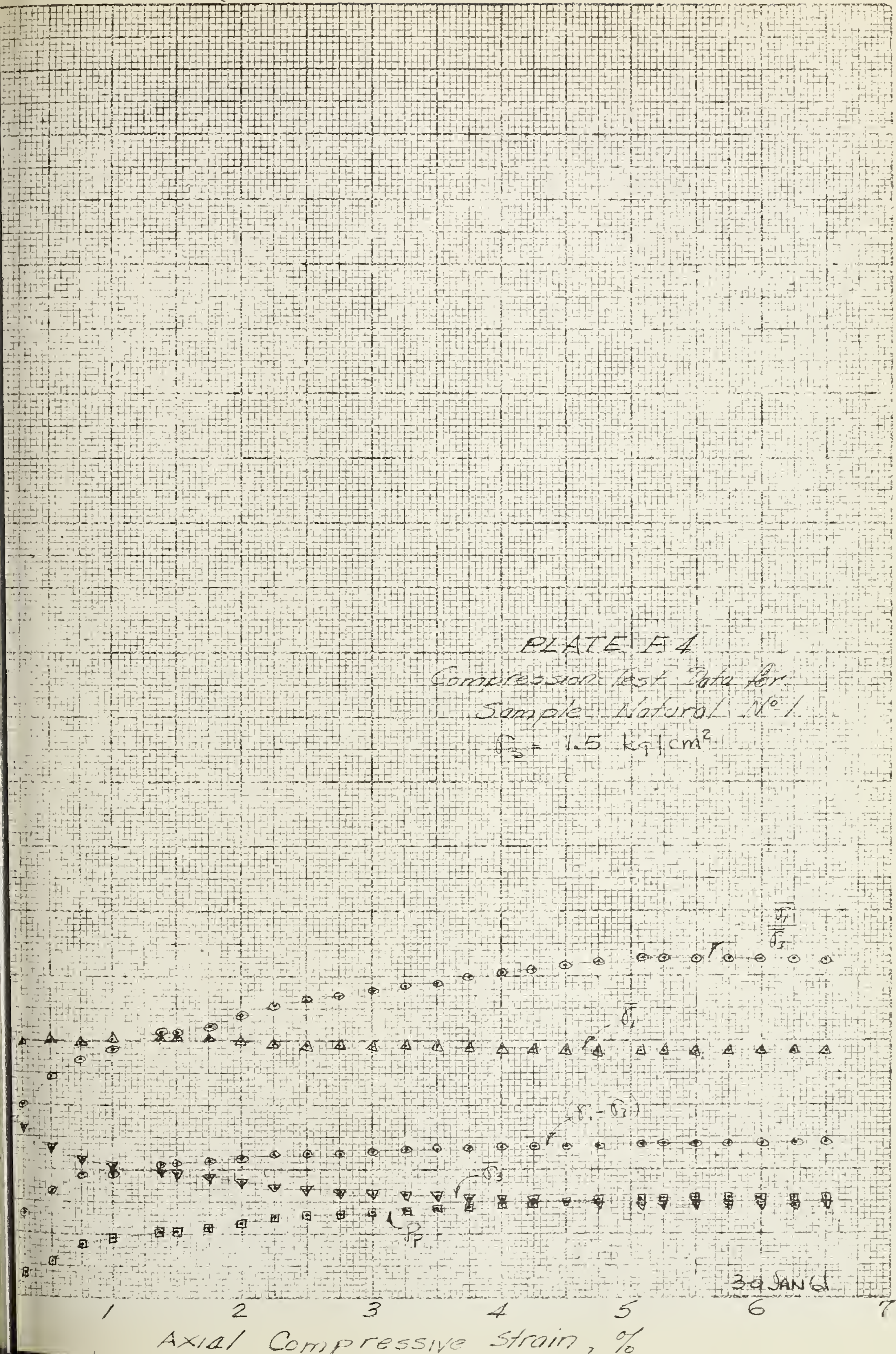


PLATE A4
Compression Test Data for
Sample Natural No 1
$\sigma_3 = 1.5$ kg/cm²

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XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Natural
Depth sample 2
Engineer ST.
Technician
Date of Set-up 25 Jan 61
Test Lateral Pressure 2.75 kg/cm²

Description

Weight 132.841 gms

mm	1. 80.12	2. 80.00	Aver: 80.06	Area Top	A_T	9.93	sq.mm
n Top	1. 35.58	2. 35.54	Aver: 35.56	Area Centre	A_C	9.83	sq.mm
entre	1. 35.34	2. 35.40	Aver: 35.37	Area Bottom	A_B	9.73	sq.mm
ttom	1. 35.20	2. 35.20	Aver: 35.20				

Volume 78.70 cc

Average X-Sect Area
$= \frac{1}{4}(A_T + 2A_C + A_B)$
$= 9.83$

IDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
00	0	24.10	
	0.1	23.05	
	0.25	22.88	
	0.5	22.63	
	1	22.30	
	2	21.75	
	3	21.40 (air)	
	4	21.00	
	6	20.30	
	8	19.55	
	10	19.00	
	15	17.90	
	20	17.00	
	30	15.60	
	40	14.50	
	60	13.20	
	80	12.30	
40	160	10.85	
0	250	10.30	
40	400	9.90	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
27 JAN 0845	13,65	8.95	
✓ 1130	1530	8.85	
1730	1890	8.65	
28 JAN 1000	2880	8.30	
29 ✓ 1140	4420	7.90	
30 ✓ 0830	5670	7.55	
✓ 2040	6400	7.25	
31 ✓ 0930	7170	7.15	
1 FEB 1710	9070	6.90	
2 FEB 1655	10,495	6.70	
3 FEB 1730	11,970	6.65	
6 FEB 0830	15,750	6.50	17.90

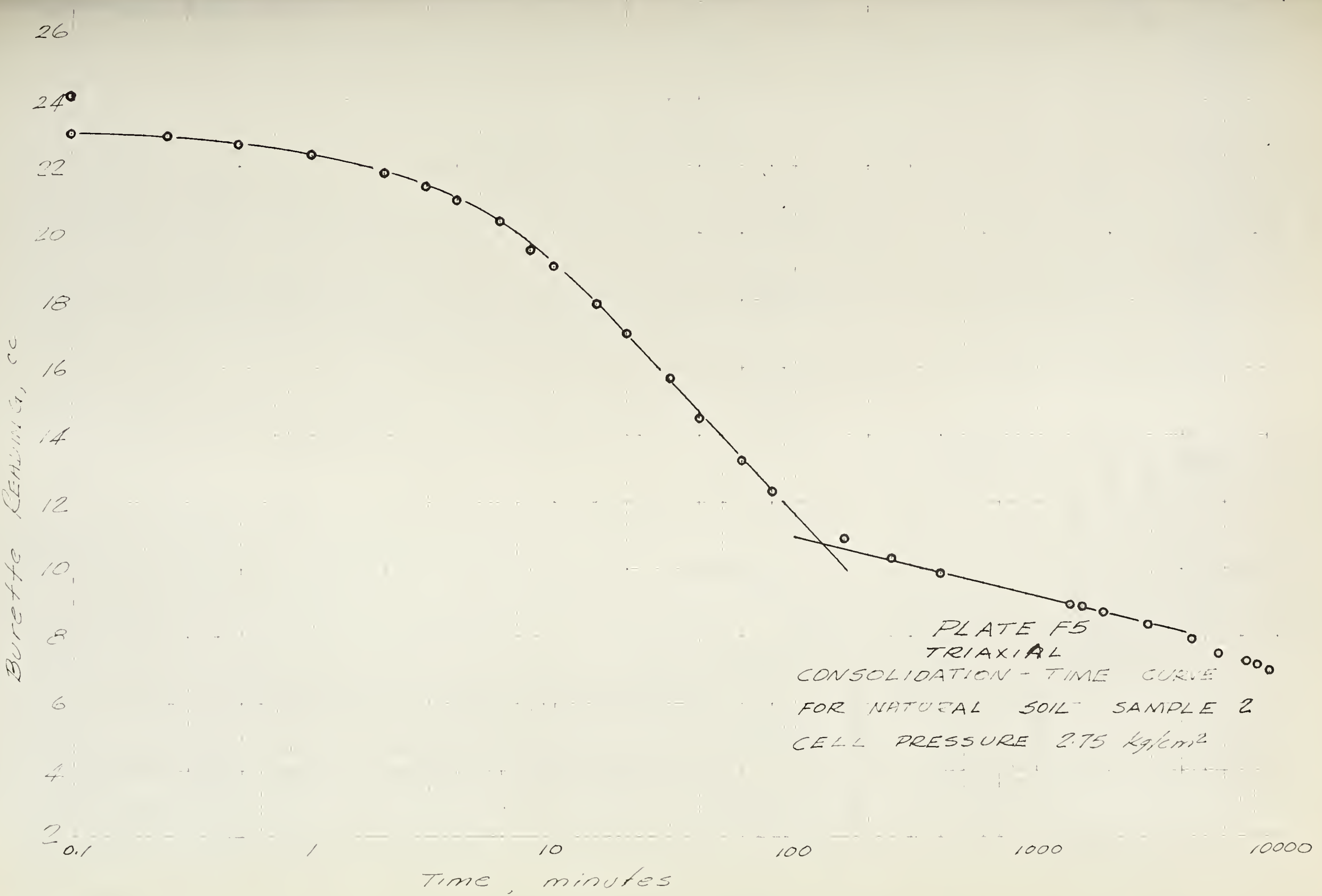


PLATE F5
TRIAXIAL
CONSOLIDATION - TIME CURVE
FOR NATURAL SOIL SAMPLE 2
CELL PRESSURE 2.75 kg/cm²

PLATE F5

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

L COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Natural
Depth ____ Sample 2
Engineer ST
Technician ____
Date of Test 6 FEB 61
Test Lateral Pressure σ_3 2.75 kg/cm²
Back Pressure 2 kg/cm²
Remarks ____
Rate of Strain 600 min/10mm
Proving Ring No 3-1498
Area Correction Factor 1.19

ailure
3) = ____ σ_1= ____
____
____
____
____ %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_p}{\sigma_1 - \sigma_3}$ %
0	9.83	0			0				0	
3	9.83	15	.0525	0.09	.01	2.83	2.74	1.03	0.038	11.1
6	9.84	35	.0514	0.22	+.07	2.90	2.68	1.08	.075	31.8
10	9.84	75	.0495	0.45	.17	3.03	2.58	1.17	.125	37.7
20	9.85	158	.0470	0.84	.37	3.22	2.38	1.35	.250	44.0
40	9.88	247	.0456	1.36	.71	3.40	2.04	1.67	.500	52.2
60	9.91	287	.0452	1.56	.90	3.41	1.85	1.84	.75	57.7
80	9.93	309	.0450	1.67	1.00	3.42	1.75	1.95	1.0	60.0
120	9.98	334	.04473	1.78	1.15	3.38	1.60	2.11	1.50	64.5
140	10.01	343	04433	1.81	1.20	3.36	1.55	2.17	1.75	66.2
160	10.03	349	.0446	1.85	1.26	3.34	1.49	2.24	2.00	68.0
180	10.06	356	.04452	1.87	1.30	3.32	1.45	2.29	2.25	69.5
200	10.09	361	.04450	1.89	1.37	3.27	1.38	2.37	2.50	72.5
220	10.11	365	.0445	1.91	1.40	3.26	1.35	2.41	2.75	73.2
240	10.14	370	.04445	1.93	1.42	3.26	1.33	2.45	3.00	73.5
260	10.16	373	.04442	1.94	1.45	3.24	1.30	2.49	3.25	78.7
280	10.19	375	.0444	1.94 *	1.48	3.21	1.27	2.53	3.50	76.2
300	10.22	3.79	.04437	1.96	1.53	3.18	1.22	2.61	3.75	78.0
320	10.25	381	.04435	1.96	1.55	3.16	1.20	2.63	4.00	79.0
340	10.27	383	.04425	1.96	1.56	3.15	1.19	2.65	4.25	79.5
360	10.30	385	04432	1.97	1.58	3.14	1.17	2.68	4.50	80.0
380	10.33	387	04430	1.97	1.59	3.13	1.16	2.70	4.75	80.6
400	10.36	388	04430	1.97	1.63	3.09	1.12	2.76	5.00	82.2
420	10.38	390	0443	1.98	1.64	3.09	1.11	2.78	5.25	82.7
440	10.41	391	0443	1.98	1.64	3.09	1.11	2.78	5.50	82.7
480	10.47	393	04427	1.98	1.66	3.07	1.09	2.82	6.00	83.7
525	10.53	395	.04425	1.98	1.69	3.04	1.06	2.87	6.56	85.2
ecimen	showing	initial	signs	of tipping	& deve	loping	an "S"	bend		

F14

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

L COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth Sample 2

Area Correction Factor 1.19

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg Div	$\sigma 1 - \sigma 3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg cm^2 P_w	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$
538	10.55	396	.04425	1.98	1.69	3.04	1.06	2.87	6.73	85.2
600	10.64	397	.04424	1.96	1.73	2.98	1.02	2.92	7.50	88.1
640	10.70	405	.04418	1.99	1.72	3.02	1.03	2.96	8.00	86.3
692	10.77	413	.04413	2.01	1.74	3.02	1.01	2.99	8.65	86.5

ntent – Initial 52.50

Final 38.29

turation – Initial 96.5

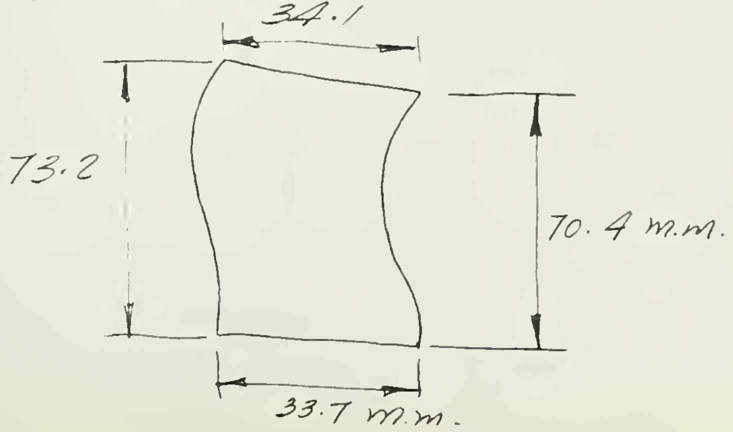
Final 97.5

– Initial 1.51

Final 1.09

e Reaction 83%

Sketch of Failure



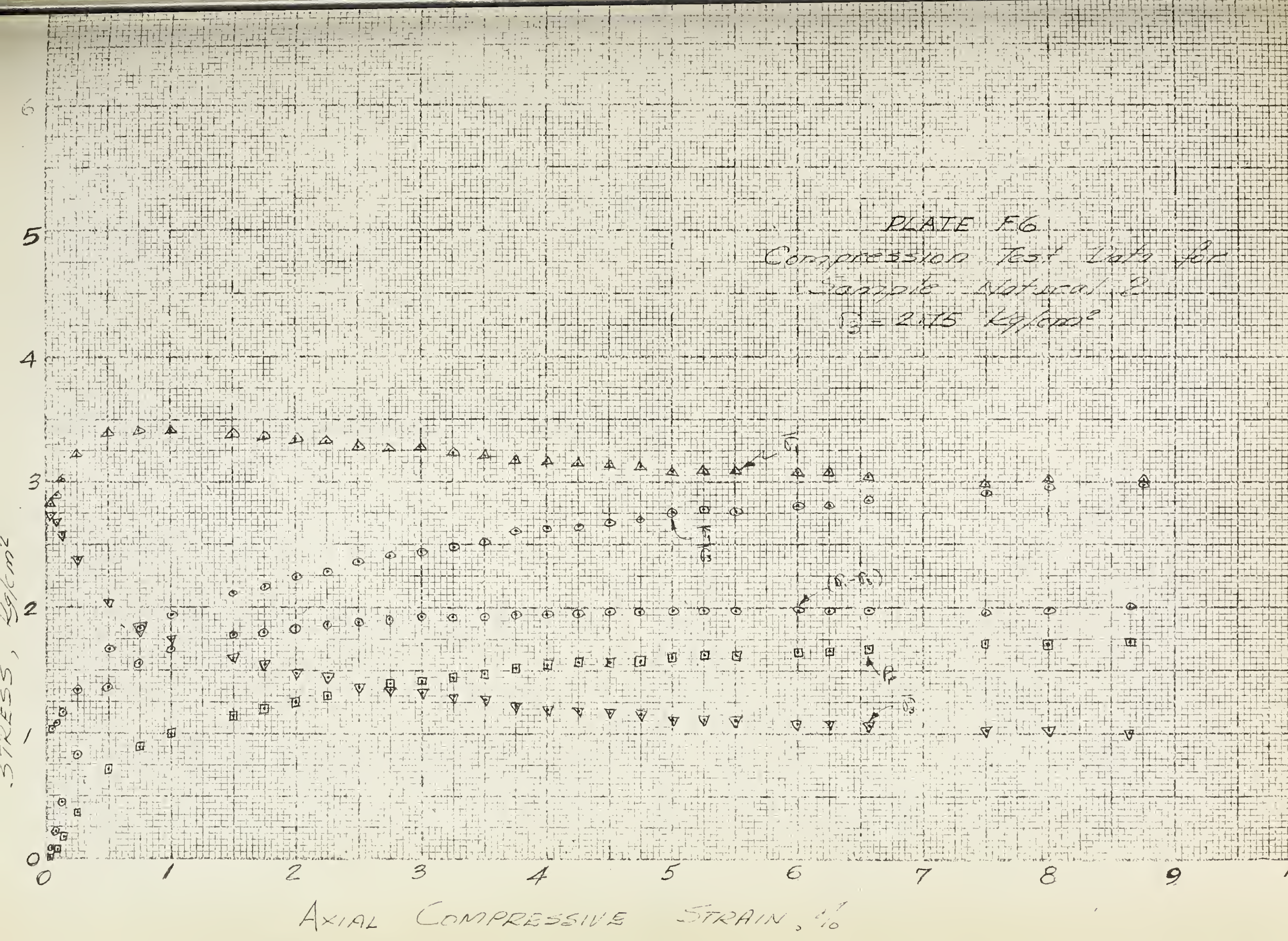


PLATE F6

Compression Test Data for Sample Natural 2

$\sigma_3 = 2.15$ kg/cm²

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Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Natural
Depth sample 3
Engineer S.T.
Technician
Date of Set-up 1 Feb 61
Test Lateral Pressure 4 kg/cm²

e Description

nternal wool wicks
Weight 135.749 gms

mm	1. 80.16	2. 80.10	Aver: 80.13	Area Top	A_T	9.731	sq.mm
n Top	1. 35.18	2. 35.22	Aver: 35.20	Area Centre	A_C	9.876	sq.mm
entre	1. 35.44	2. 35.48	Aver: 35.46	Area Bottom	A_B	9.931	sq.mm
ttom	1. 35.58	2. 35.54	Aver: 35.56	Average X-Sect Area			

Volume 78.93 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

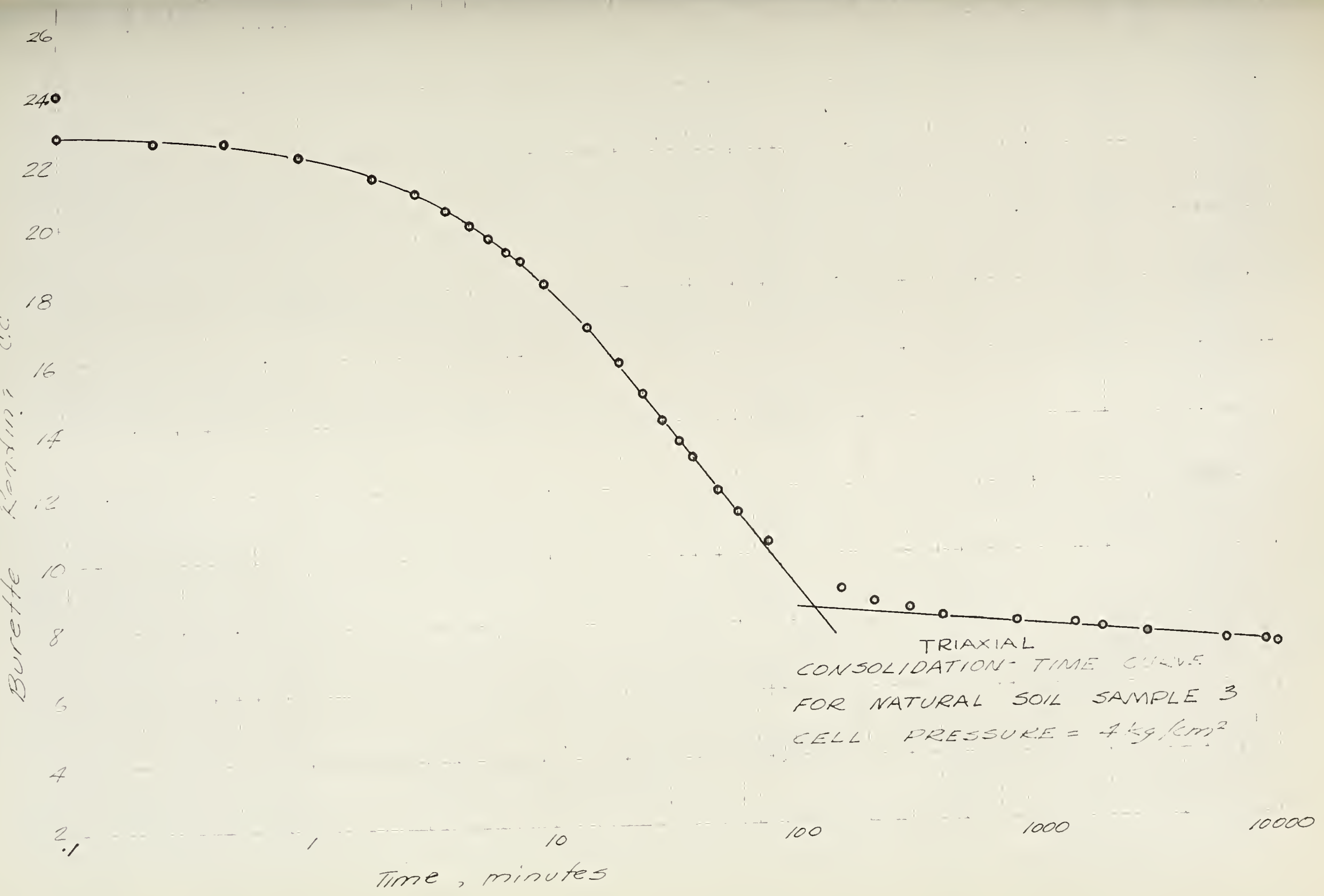
$= 9.85$

IDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
00	0	24.00	
	0.1	22.7	
	0.25	22.5	
	0.5	22.45	
	1.0	22.0	
	2	21.3	
	3	20.8	
	4	20.3	
	5	19.85	
	6	19.45	
	7	19.10	
	8	18.75	
	10	18.10	
	15	16.8	
	20	15.7	
	25	14.75	
	30	14.00	
	35	13.40	
	40	12.85	
	50	11.95	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
	60	11.20	
	80	10.35	
	158	8.95	
	215	8.55	
	300	8.30	
1650	410	8.10	
3 Feb 0001	840	7.85	
1025	1465	7.70	
1735	1895	7.60	
4 Feb 0925	2845	7.50	
6 Feb 0830	6125	7.30	
8 Feb 1240	8800	7.15	
9 Feb 1020	10,100	7.10	
11 Feb 1800	13,440	7.02	
13 Feb 1010	15,850	7.05	
18 Feb 0930	23,010	7.03	
20 Feb 1030		7.03	16.97



TRIAXIAL
CONSOLIDATION - TIME CURVE
FOR NATURAL SOIL SAMPLE 3
CELL PRESSURE = 4 kg/cm²

PLATE F7

Sheet 1 F18

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

L COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth Sample 3

Engineer ST

Technician

Date of Test 20 Feb 61

Test Lateral Pressure σ_3 4 kg/cm²

Back Pressure 2 kg/cm²

Remarks

Proving Ring N° 3-1584

Rate of strain 600 min. per 10 mm

Area Correction Factor 1/8

Failure

σ_3) = 3.32 σ_1 = 7.32

1.57

5.75

2.43

9.75 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_p}{\sigma_1 - \sigma_3}$
0	9.85	0	0		-.20				0	
5	9.85	20	.0974	.24	-.24	4.48	4.24	1.06	.06	
7	9.86	30	.0964	.34	-.16	4.50	4.16	1.08	.09	
10	9.86	40	.0956	.46	-.15	4.61	4.15	1.11	.13	
14	9.87	60	.0941	.67	-.05	4.72	4.05	1.17	.18	
19	9.87	80	.0931	.89	+.04	4.85	3.96	1.22	.24	4.4
25	9.88	105	.0924	1.16	+.16	5.00	3.84	1.30	.31	13.8
30	9.89	123	.0921	1.35	+.25	5.10	3.75	1.36	.38	18.5
40	9.90	141	.0918	1.54	+.40	5.14	3.60	1.43	.50	26.0
50	9.91	167	.0914	1.82	+.57	5.25	3.43	1.53	.625	31.5
60	9.93	180	.0912	1.95	.68	5.27	3.32	1.59	.75	34.8
80	9.95	197	.0909	2.12	.77	5.35	3.23	1.66	1.00	36.3
100	9.98	210	.0908	2.25	.86	5.39	3.14	1.72	1.25	38.2
120	10.00	221	.0906	2.36	.99	5.37	3.01	1.78	1.50	41.9
140	10.03	229	.0905	2.44	1.04	5.40	2.96	1.82	1.75	42.6
160	10.05	234	.0904	2.48	1.09	5.39	2.91	1.85	2.00	43.9
180	10.08	241	.0903	2.55	1.16	5.39	2.84	1.90	2.25	45.5
200	10.11	247	.0902	2.60	1.19	5.41	2.81	1.93	2.50	45.8
220	10.13	253	.0901	2.66	1.24	5.42	2.76	1.96	2.75	46.6
240	10.16	258	.0900	2.~~6~~70	1.27	5.43	2.73	1.99	3.00	47.0
260	10.18	262	.0900	2.73	1.29	5.44	2.71	2.01	3.25	47.2
280	10.21	267	.0899	2.77	1.32	5.45	2.68	2.03	3.50	47.6
300	10.24	271	.0898	2.80	1.35	5.45	2.65	2.06	3.75	48.2
320	10.26	276	.0897	2.85	1.37	5.~~8~~48	2.63	2.08	4.00	48.0
340	10.29	280	.0896	2.88	1.38	5.50	2.62	2.10	4.25	47.9
380	10.35	287	.0895	2.93	1.37	5.56	2.63	2.11	4.75	46.7
420	10.40	294	.0894	2.98	1.37	5.~~56~~61	2.63	2.13	5.25	46.0
440	10.43	298	.0894	3.01	1.38	5.63	2.62	2.15	5.50	45.8
500	10.52	309	.0892	3.09	1.38	5.71	2.62	2.18	6.25	44.6

Sheet 2

F19

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth Sample 3

Area Correction Factor 1.18

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$
520	10.54	312	.0891	3.11	1.36	5.75	2.64	2.18	6.50	43.6
600	~~[illegible]~~ 10.66	326	.0889	3.21	1.29	5.92	2.71	2.18	7.50	40.2
620	10.69	330	.0888	3.23	1.32	5.91	2.68	2.21	7.75	40.8
642	10.72	332	.0887	3.26	1.27	5.99	2.73	2.19	8.02	39.0
660	10.75	338	.0887	3.29	1.~~[illegible]~~39	5.90	2.61	2.26	8.25	42.2
700	10.81	341	.0886	3.30	1.48	5.82	2.52	2.31	8.75	44.8
7~~02~~30	10.84	343	.0886	3.31	1.52	5.79	2.48	2.33	9.00	46.9
740	10.87	344	.0886	3.31	1.51	5.80	2.49	2.32	9.25	45.5
760	10.90	345	.0886	3.31	1.59	5.72	2.41	2.37	9.50	48.0
780	10.93	347	.0885	3.32	1.57	5.75	2.43	2.37	9.75	47.2 ←
804	10.97	349	.0885	3.32	1.58	5.74	2.42	2.37	10.05	47.5

ontent - Initial 51.14

Final 34.43

aturation - Initial 98.5

Final 99.2 ~~0.964~~

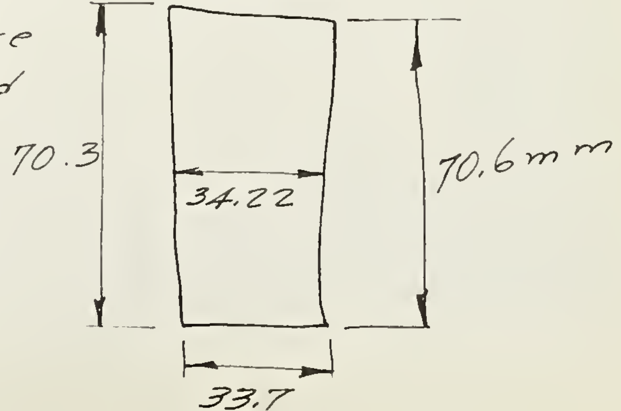
- Initial 1.44

Final 0.964

ure Reaction 74%

Sketch of Failure

some evidence of "S" shaped distortion



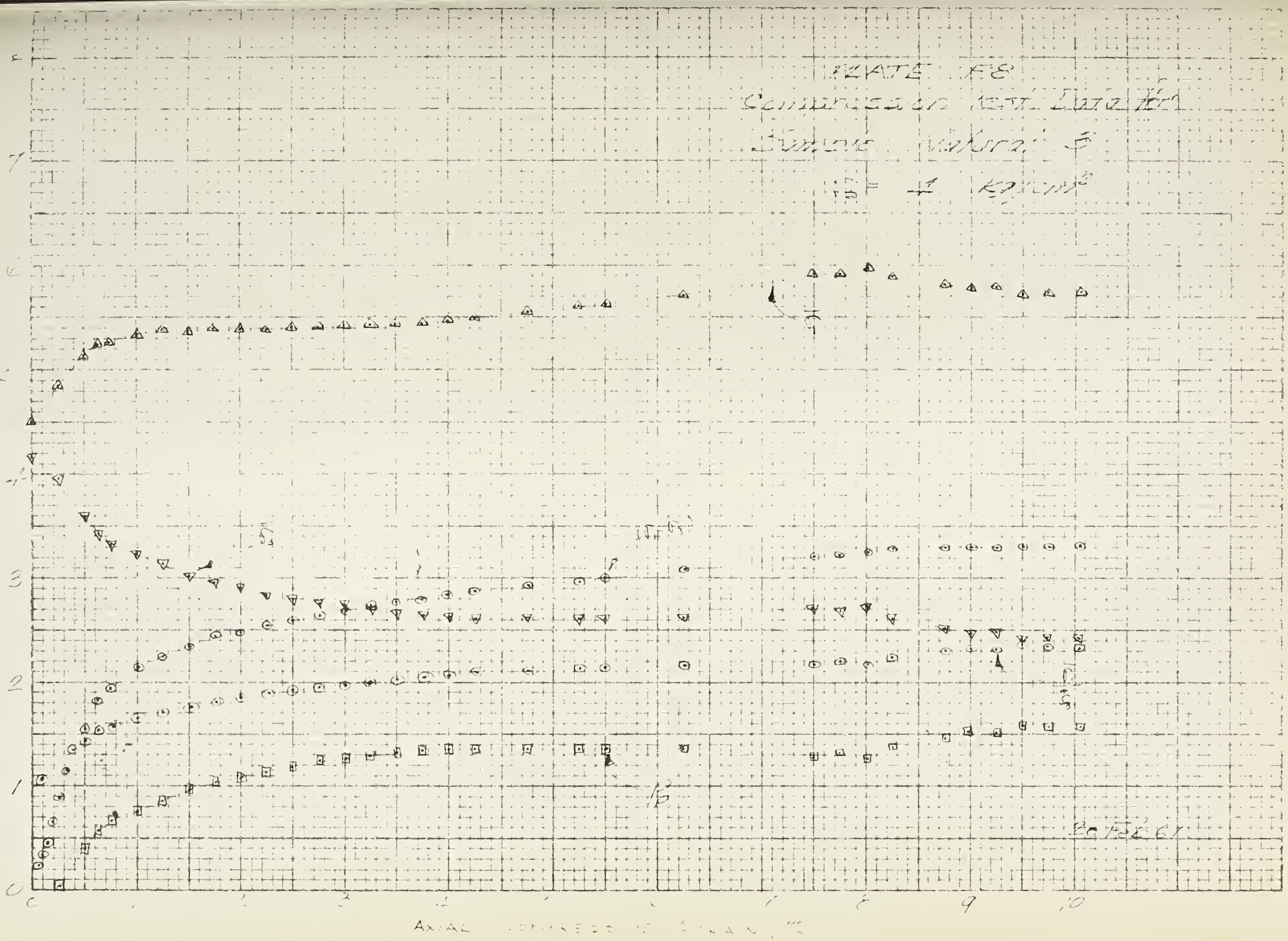


PLATE F8
Compression Test Data for
Sample Natural F
σ3 = 4 kg/cm²
LOAD (kg/cm)
AXIAL COMPRESSIVE STRAIN, %
0
1
2
3
4
5
6
7
8
9
10
26 Feb 67

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Natural
Depth sample 4
Engineer S.T.
Technician DW
Date of Set-up 27 Oct 61
Test Lateral Pressure 4 kg/cm²

Description

epeat Test.

Weight 135.551 gms

mm	1. 80.50	2. 80.56	Aver: 80.53	Area Top A_T 10.044	Sq.mm
Top	1. 35.80	2. 35.72	Aver. 35.76	Area Centre A_C 9.954	sq.mm
ntre	1. 35.60	2. 35.60	Aver: 35.60	Area Bottom A_B 9.842	sq.mm
ttom	1. 35.40	2. 35.40	Aver: 35.40	Average X-Sect Area	

olume 80.13 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$=$ 9.95mm²

DATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
40	0	24.85	
	.1	23.75	
	.25	23.50	
	.50	23.15	
	1	22.65	
	2	21.85	
	3	21.20	
	4	21.05	
	6	19.65	
	8	18.90	
	10	18.10	
	16	16.60	
	20	15.40	
	25	14.40	
	30	13.60	
	35	12.95	
	40	12.35	
	45	11.88	
	50	11.40	
	60	10.75	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
27 Oct	70·[illegible]	10.25	
	80	9.85	
	90	9.60	
	100	9.30	
	120	9.00	
	150	8.65	
	180	8.40	
	200	8.30	
	250	8.15	
1745	365	7.95	
2010	510	7.80	
28 Oct 0810	1230	7.60	
29 Oct 1530	3110	7.20	17.65

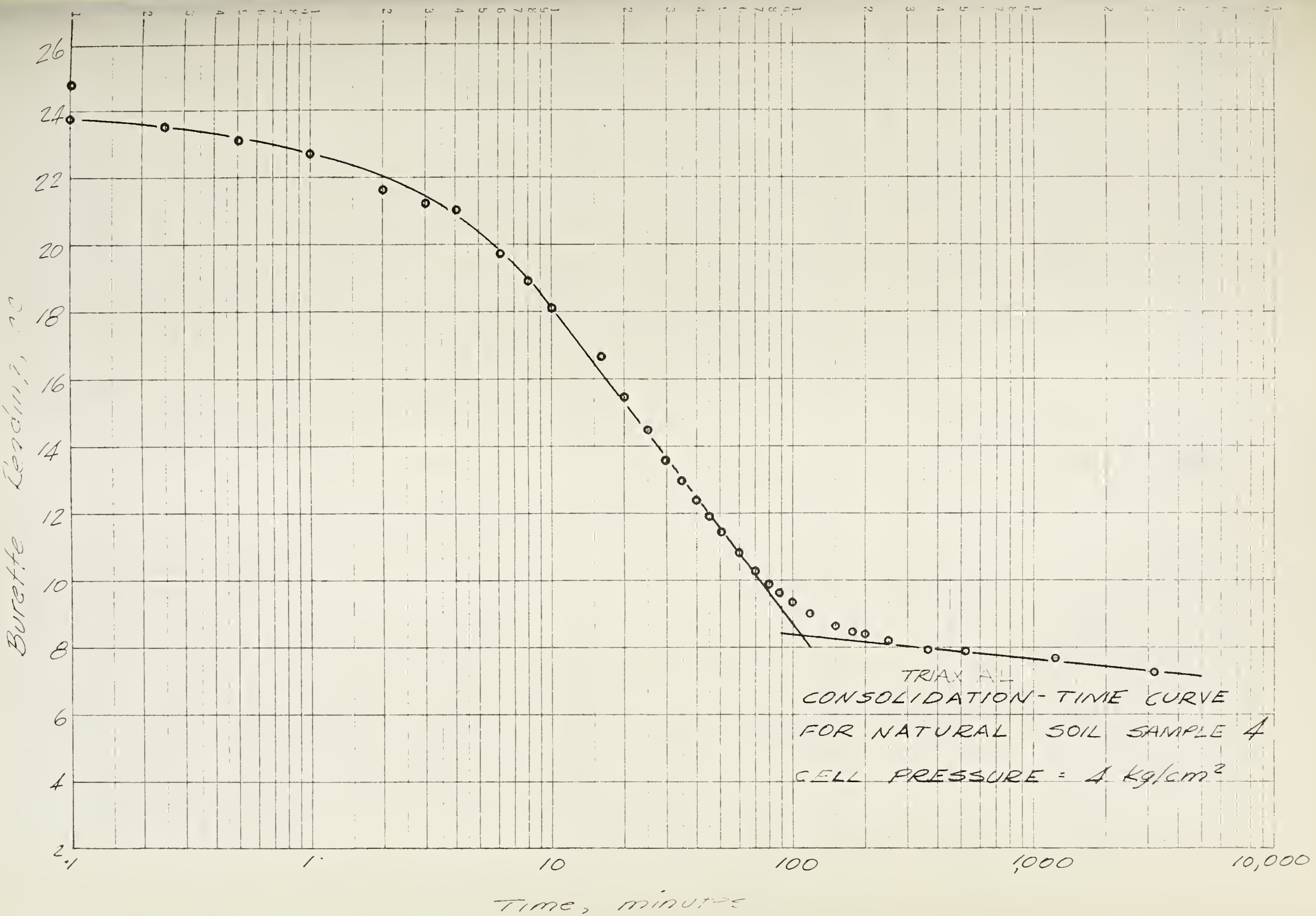

TRIAX A-1
CONSOLIDATION-TIME CURVE
FOR NATURAL SOIL SAMPLE 4
CELL PRESSURE = 4 kg/cm²
Burette Reading, cc
26
24
22
20
18
16
14
12
10
8
6
4
2
.1
1
10
100
1,000
10,000
Time, minutes

PLATE F9

Sheet 1 F23

UNIVERSITY OF ALBERTA

)epartment of Civil Engineering

Soil Mechanics Laboratory

COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth Sample 4

Engineer ST

Technician DW

Date of Test 30 Oct 61

Test Lateral Pressure σ_3 4 kg/cm²

Back Pressure 2 ✓

Remarks

Rate of Strain 362.3 min per 10mm

Proving Ring N° 3-1498

Area Correction Factor 1.186

ilure

$_3$) = 2.81 σ_1 = 6.81

2.40 kg/cm²

1.41 ✓

1.60

13.5 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\overline{A}$ Pp / $\sigma_1 - \sigma_3$ %
						$\overline{\sigma}_1$ Major	$\overline{\sigma}_3$ Minor	$\overline{\sigma}_1 / \overline{\sigma}_3$		
0	9.95	0		0						
2	✓	~~.05~~ 20	.0525	.125	+.11				.025	
3.5	✓	40	.0511	.244	+.15				.043	
5	9.955	60	.0501	.358	.19				.062	
8	9.96	105	.04844	.61	.28	4.33	3.72	1.164	.1	45.9
16	9.97	195	.04635	1.08	.47	4.61	3.53	1.306	.2	43.5
24	9.98	255	.04553	1.38	.64	4.74	3.36	1.411	.3	49.0
32	9.99	294	.04508	1.57	.77	4.80	3.23	1.486	.4	49.0
40	10.00	320	.04486	1.70	.88	4.82	3.12	1.544	.5	51.7
48	10.01	343	.04462	1.81	.97	4.84	3.03	1.597	.6	53.6
56	10.02	360	.04451	1.90	1.05	4.85	2.95	1.644	.7	55.3
64.5	10.03	375	.04440	1.97	1.10	4.87	2.90	1.679	.8	55.8
72.5	10.04	387	.04430	2.03	1.19	4.84	2.81	1.722	.9	58.6
80.5	10.05	399	.04422	2.08	1.25	4.83	2.75	1.756	1.0	60.1
88.5	10.06	407	.04416	2.12	1.30	4.82	2.70	1.785	1.1	61.3
96.5	10.07	415	.04410	2.16	1.35	4.81	2.65	1.815	1.2	62.5
104.5	10.08	421	.04407	2.18	1.40	4.78	2.60	1.838	1.3	
112.5	10.09	428	.04401	2.21	1.44	4.77	2.56	1.863	1.4	65.2
121	10.10	435	.04398	2.25	1.48	4.77	2.52	1.893	1.5	
129	10.11	441	.04394	2.27	1.51	4.76	2.49	1.912	1.6	66.5
137	10.12	446	.04391	2.30	1.55	4.75	2.45	1.939	1.7	67.4
145	10.13	451	.04389	2.32	1.59	4.73	2.41	1.963	1.8	68.5
153	10.14	456	.04386	2.34	1.62	4.72	2.38	1.983	1.9	69.2
161	10.15	460	.04383	2.36	1.65	4.71	2.35	2.004	2.0	69.9
177	10.17	467	.04380	2.39	1.70	4.69	2.30	2.039	2.2	71.1
193	10.195	476	.04374	2.42	1.72	4.70	2.28	2.061	2.4	71.1
209	10.215	483	.04370	2.45	1.80	4.65	2.20	2.114	2.6	73.5
225.5	10.235	489	.04366	2.47	1.83	4.64	2.17	2.138	2.8	74.1
241.5	10.26	495	.04363	2.50	1.87	4.63	2.13	2.174	3.0	74.8

Sheet 2

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS F24

Hole No. Natural

Depth Sample 4

Area Correction Factor 1.186

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$ %
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
282	10.31	509	.04355	2.55	1.96	4.59	2.04	2.250	3.5	76.7
322	10.365	520	.04350	2.59	2.02	4.57	1.98	2.308	4.0	78.0
362.5	10.42	530	.04344	2.62	2.09	4.53	1.91	2.372	4.5	79.8
402.5	10.47	540	.04340	2.65	2.13	4.52	1.87	2.417	5.0	80.4
443	10.53	548	.04336	2.68	2.18	4.50	1.82	2.473	5.5	81.3
483	10.585	556	.04332	2.70	2.21	4.49	1.79	2.508	6.0	81.9
523.5	10.64	564	.04329	2.72	2.24	4.48	1.76	2.545	6.5	82.4
564	10.70	571	.04326	2.74	2.27	4.47	1.73	2.584	7.0	82.8
604	10.755	578	.04323	2.76	2.30	4.46	1.70	2.624	7.5	83.3
644	10.815	585	.04320	2.77	2.31	4.46	1.69	2.639	8.0	83.4
684.5	10.875	590	.04318	2.78	2.33	4.45	1.67	2.665	8.5	83.8
725	10.935		missed	Rdg					9.0	
765	10.995	602	.04312	2.80	2.36	4.44	1.64	2.707	9.5	84.3
805	11.055	606	.04311	2.80	2.36	4.44	1.64	2.707+	10.0	84.3
845.5	11.12	611	.04310	2.81	2.37	4.44	1.63	2.724	10.5	84.3
886	11.18	615	.04308	2.81	2.37	4.44	1.63	2.724	11.0	84.3
926	11.24	619	.04307	2.81	2.38	4.43	1.62	2.735	11.5	84.7
966	11.31	623	.04305	2.81	2.39	4.42	1.61	2.745	12.0	85.1
1007	11.37	628	.04302	2.82	2.39	4.43	1.61	2.751	12.5	84.8
1047	11.435	629	.04302	2.81	2.39	4.42	1.61	2.745	13.0	85.1
1087	11.50	633	.04300	2.81	2.40	4.41	1.60	2.756	13.5	85.4 ←
1127.5	11.57	636	.04300	2.80	2.40	4.40	1.60	2.750	14.0	85.7
1168.	11.64	639	.04299	2.80	2.40	4.40	1.60	2.750	14.5	85.7
1208	11.705	641	.04298	2.79	2.40	4.39	1.60	2.744	15	86.0

ontent – Initial 52.0

Final 36.56

aturation – Initial 96.7 %

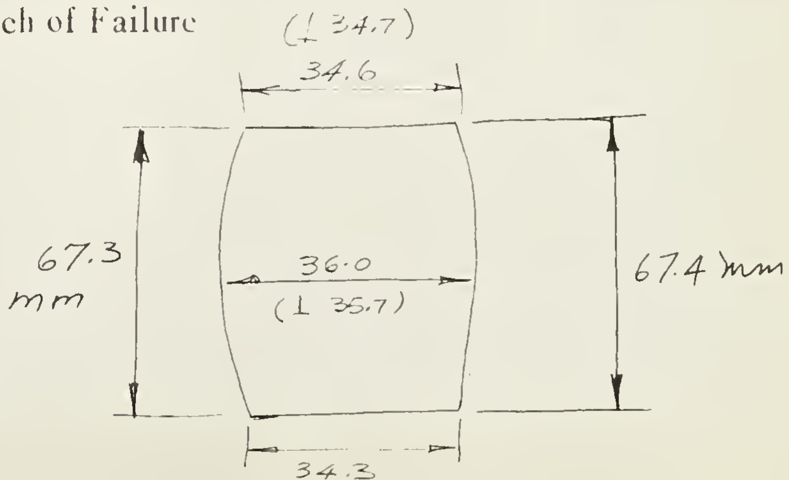
Final 98.1

– Initial 1.499

Final 1.036

ure Reaction ≈85 %

Sketch of Failure



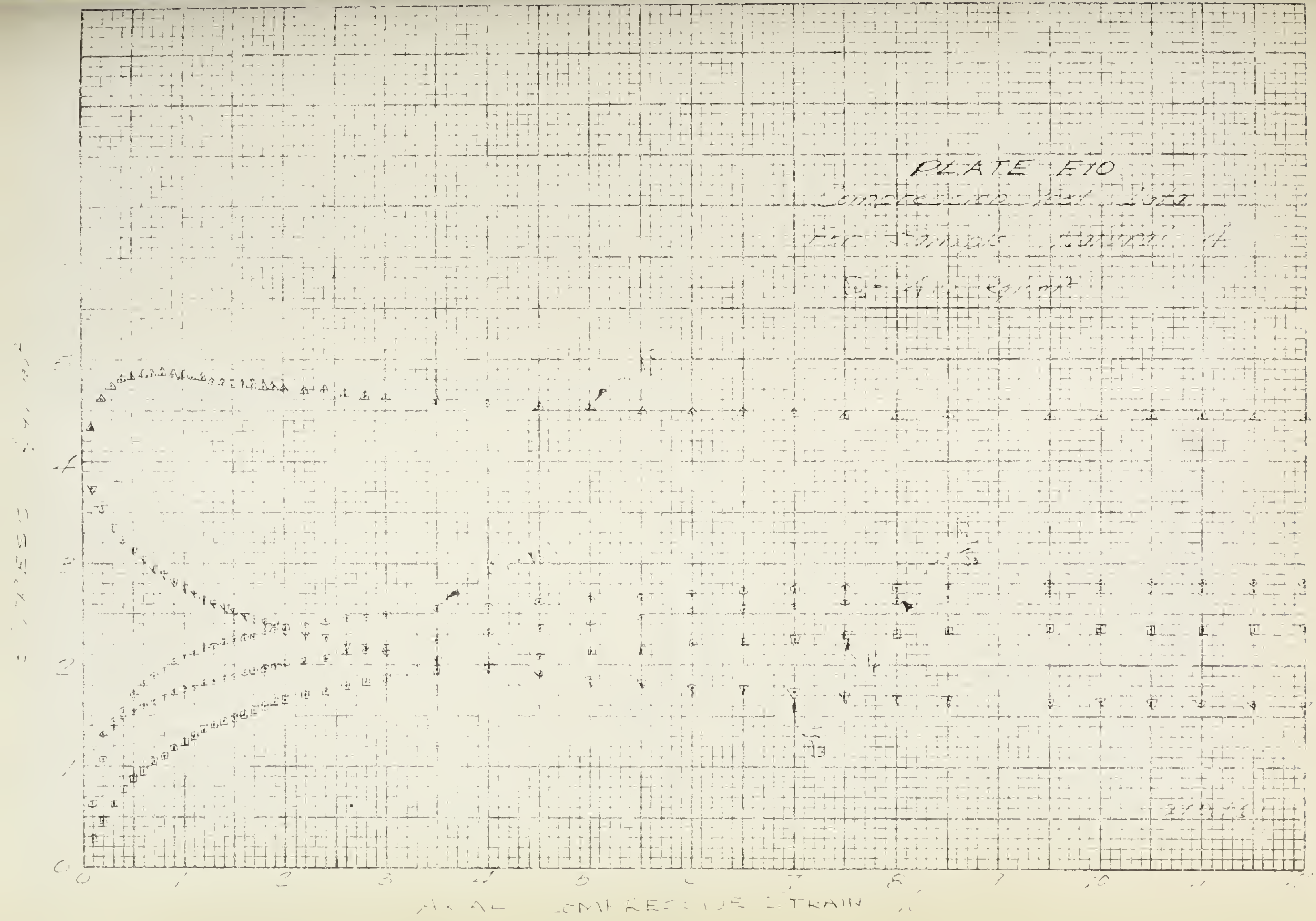


PLATE E10

[illegible]

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth sample 5

Engineer ST

Technician D.W

Date of Set-up 13 JUN 61

Test Lateral Pressure 5.25

e Description

nternal wicks

Weight 135.369 gms

mm	1. 80.60	2. 80.56	Aver: 80.58	Area Top	A_T 9.870 sq.mm
n Top	1. 35.30	2. 35.60	Aver: 35.45	Area Centre	A_C 9.976 sq.mm
entre	1. 35.60	2. 35.68	Aver: 35.64	Area Bottom	A_B 10.134 sq.mm
ottom	1. $3\overset{5}{\cancel{6}}.94$	2. 35.90	Aver: 35.92	Average X-Sect Area	

Volume 80.50 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 9.99$

IDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
45	0	24.70	
	·1	23.90	
	·25	23.65	
	·5	23.45	
	1.0	23.00	
	2	22.30	
	4	21.15	
	6	20.30	
	8	19.50	
	10	18.90	
	15	17.40	
	20	16.40	
	30	14.35	
	61	10.60	
	100	8.35	
	203	6.60	
	460	5.95	
15	1140	5.80	
5	1590	5.75	18.95

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.

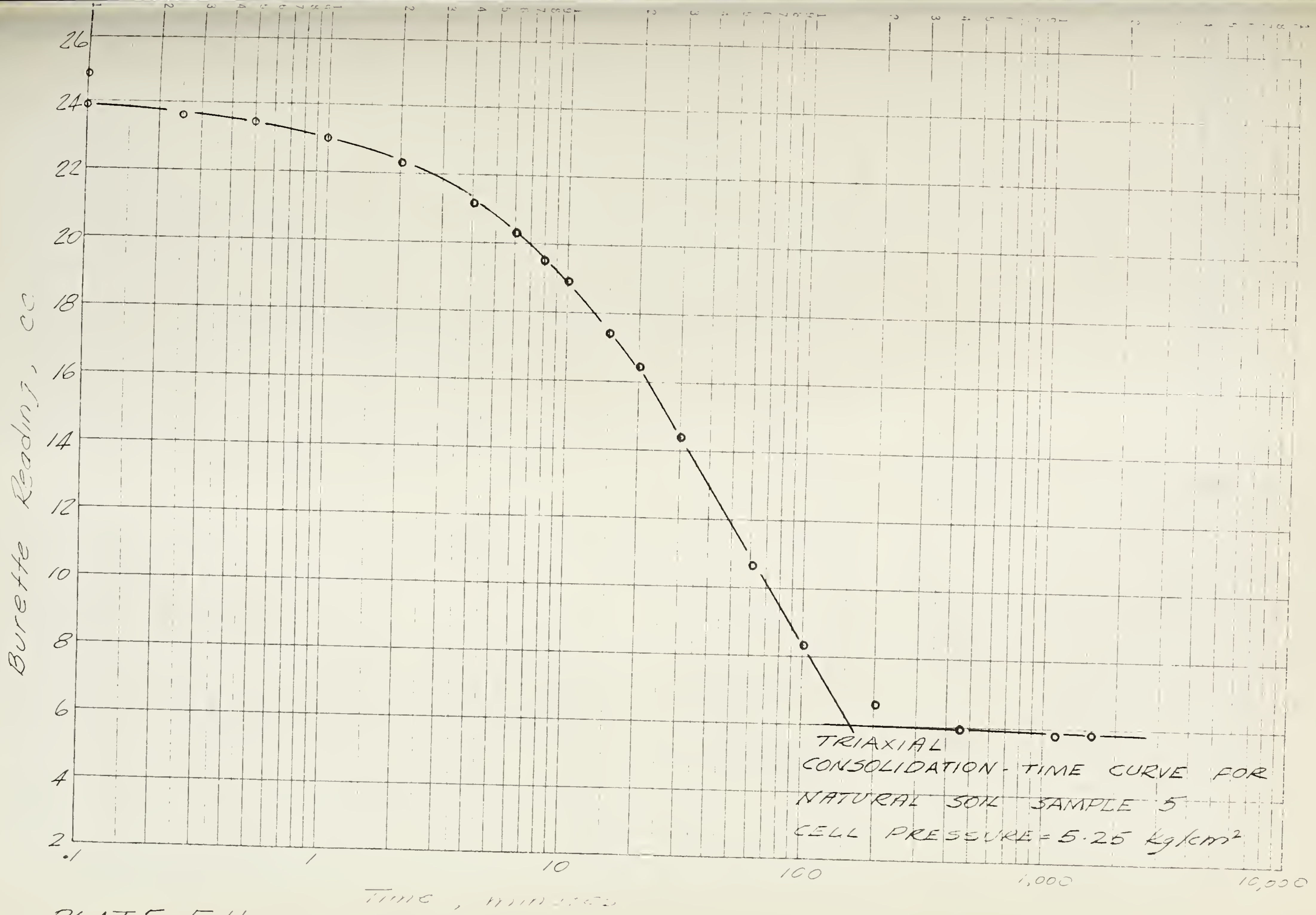

TRIAXIAL
CONSOLIDATION - TIME CURVE FOR
NATURAL SOIL SAMPLE 5
CELL PRESSURE = 5·25 kg/km²
Burette Reading, cc
26
24
22
20
18
16
14
12
10
8
6
4
2
·1
1
10
100
1,000
10,000
Time, minutes

PLATE F11

Sheet 1 F28

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

L COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth ______ Sample 5

Engineer ST

Technician DW

Date of Test 15 JUN 61

Test Lateral Pressure σ_3 5.25 Kg/cm²

Back Pressure 2.0 Kg/cm²

Remarks

Proving Ring N° 3 - 1498

Rate of Strain 600 min/10mm

Area Correction Factor 1.20

Failure

$\sigma_3) =$ ______ $\sigma_1 =$ ______

______

______

______

______ %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\frac{\bar{A}\ P_P}{\sigma_1 - \sigma_3}$ %
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
0	9.99	0	0						0	
1	9.99	20	.0525	.13	+.03				.01	
2	9.99	30	.0517	.19	+.03				.025	
3	✓	40	.0511	.25	.04				.037	
3.5	✓	50	.0506	.30	.06				.043	
4.5	10.00	70	.0497	.42	.10	5.57	5.15	1.08	.056	23.8
8	✓	131	.04768	.75	.26	5.74	4.99	1.15	.10	34.7
16	10.01	234	.0458	1.28	.55	5.98	4.70	1.27	.20	43.0
24	10.02	290	.04515	1.57	.74	6.08	4.51	1.35	.30	47.1
32	10.03	327	.0448	1.75	.88	6.12	4.37	1.40	.40	50.3
40	10.04	354	.04456	1.89	1.00	6.14	4.25	1.44	.50	52.9
48	10.05	375	.04440	1.99	1.11	6.13	4.14	1.48	.60	58.8
56	10.06	394	.04427	2.08	1.23	6.10	4.02	1.52	.70	59.1
64.5	10.07	412	.04413	2.17	1.33	6.09	3.92	1.55	.80	61.3
72.5	10.08	425	.04404	2.23	1.39	6.09	3.86	1.60	.90	62.3
80.5	10.09	437	.04398	2.29	1.47	6.07	3.78	1.61	1.00	64.1
88.5	10.10	448	.04390	2.34	1.53	6.06	3.72	1.63	1.10	65.3
96.5	10.11	458	.04383	2.38	1.61	6.02	3.64	1.65	1.20	67.6
104.5	10.12	467	.04379	2.42	1.66	6.01	3.59	1.67	1.30	68.5
113	10.13	476	.04372	2.47	1.72	6.00	3.53	1.70	1.40	69.6
121	10.14	485	.04368	2.51	1.75	6.01	3.50	1.72	1.50	69.7
129	10.15	491	.04366	2.53	1.80	5.98	3.45	1.73	1.60	71.1
137	10.165	498	.04360	2.56	1.84	5.97	3.41	1.75	1.70	71.9
145	10.175	505	.04357	2.59	1.90	5.94	3.35	1.77	1.80	73.4
153	10.185	513	.04353	2.63	1.92	5.96	3.33	1.79	1.90	73.0
161	10.195	519	.04350	2.66	1.98	5.93	3.27	1.81	2.00	74.4
169	10.205	524	.04347	2.68	2.01	5.92	3.24	1.83	2.10	75.0
177	10.22	530	.04343	2.70	2.05	5.90	3.20	1.84	2.20	75.9
185	10.23	535	.04341	2.72	2.08	5.89	3.17	1.86	2.30	76.5

Sheet 2

F29

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth

Sample 5

Area Correction Factor 1.20

Strain Dial Div.	A_c cm2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm2 P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$ %
193	10.24	541	.04340	2.75	2.10	5.90	3.15	1.87	2.40	76.4
201.5	10.245	546	.04337	2.77	2.14	5.88	3.11	1.89	2.50	77.2
209.5	10.255	551.5	.04335	2.80	2.18	5.87	3.07	1.91	2.60	77.8
217.5	10.27	554.5	.04333	2.81	2.18	5.88	3.07	1.92	2.70	77.5
225.5	10.28	5~~4~~59	.04330	2.83	2.20	5.88	3.05	1.93	2.80	77.7
233.5	10.29	563.5	.04330	2.85	2.25	5.85	3.00	1.95	2.90	78.9
242	10.30	568	.04328	2.86	2.26	5.85	2.99	1.96	3.00	79.0
258	10.32	577	.04323	2.90	2.29	5.86	2.96	1.98	3.20	78.9
274	10.34	584	.04320	2.93	2.36	5.82	2.89	2.01	3.40	80.5
290	10.365	590	.04318	2.95	2.37	5.83	2.88	2.02	3.60	80.3
306	10.38	596	.04315	2.97	2.39	5.83	2.86	2.04	3.80	80.5
322	10.41	604	.04311	3.00	2.42	5.83	2.83	2.06	4.00	80.6
338	10.43	610	.04310	3.02	2.49	5.78	2.76	2.09	4.20	82.5
354.5	10.45	618	.04306	3.06	2.55	5.76	2.70	2.13	4.40	83.3
370.5	10.475	623	.04304	3.07	2.57	5.75	2.68	2.15	4.60	83.7
386.5	10.50	630	.04301	3.10	2.63	5.72	2.62	2.18	4.80	84.8
403	10.52	636	.04300	3.12	2.64	5.73	2.61	2.20	5.00	84.6
443	10.575	652	.04295	3.18	2.73	5.70	2.52	2.26	5.50	85.8
483.5	10.63	660	.04290	3.20	2.76	5.69	2.49	2.29	6.00	86.2
524	10.69	671	.04287	3.23	2.79	5.69	2.46	2.31	6.50	86.4
564	10.75	680	.04284	3.25	2.84	5.66	2.41	2.35	7.0	87.3
604.5	10.805	689	.04280	3.27	2.83	5.69	2.42	2.35	7.5	86.5
725	10.985	713.5	.04271	3.33	2.94	5.64	2.31	2.44	9.00	88.3
741	11.01	716	.04270	3.33	2.95	5.63	2.30	2.45	9.20	88.6
765.5	11.05	720	.04268	3.34	2.95	5.64	2.30	2.45	9.50	88.3
806	11.11	727.5	.04265	3.35	2.98	5.62	2.27	2.48	10.00	89.0
846	11.17	734	.0426~~4~~3	3.36	2.96	5.65	2.29	2.47	10.50	88.0
886	11.235	739	.04261	3.36	3.00	5.61	2.25	2.49	11.00	89.3
906.5	11.27	743	.04260	3.37	2.99	5.63	2.26	2.49	11.25	88.7

ontent – Initial 54.11

Final 35.36

aturation – Initial 97.2

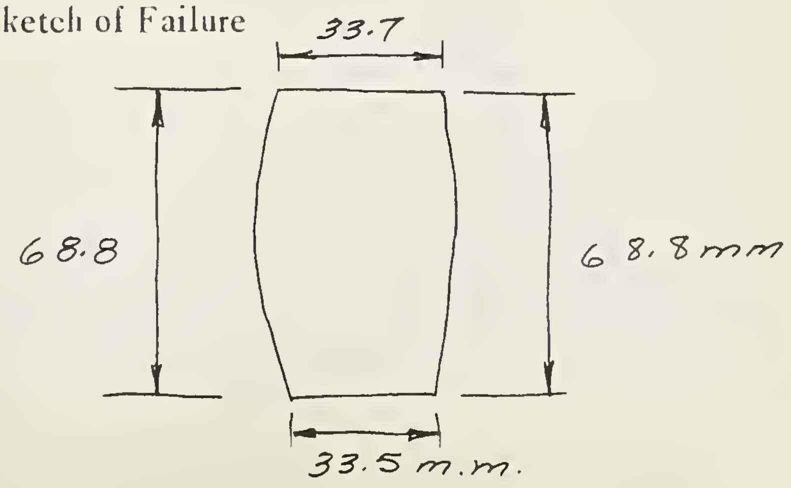
Final 97.0

– Initial 1.547

Final 1.014

re Reaction 88%

Sketch of Failure



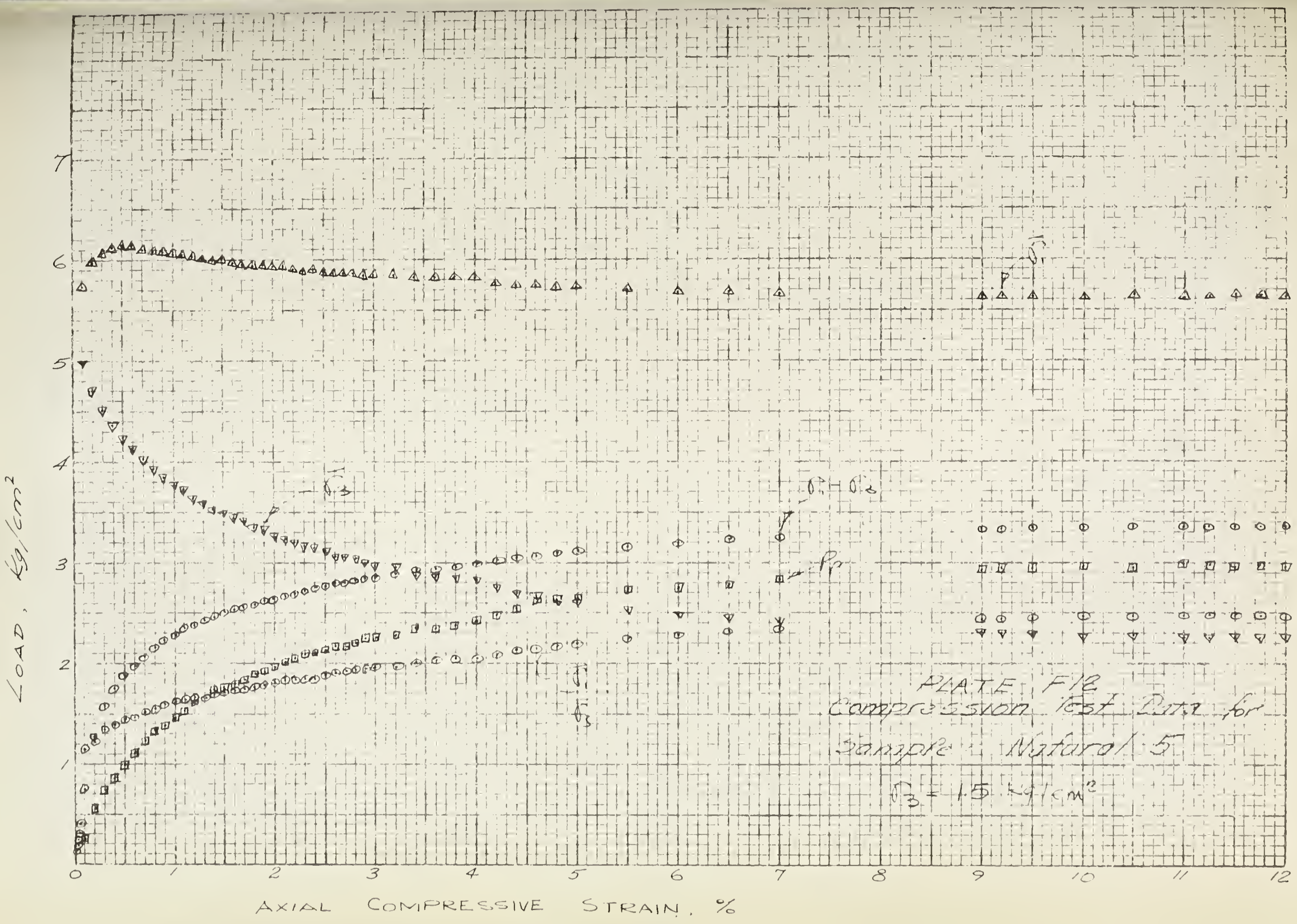


PLATE F12
Compression Test Run for
Sample Natural 5
$\sigma_3 = 1.5$ kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

le Description

internal Wicks

l Weight 137.059 gms

Project THESIS
Hole No. Natural
Depth sample 6
Engineer ST
Technician DW
Date of Set-up 13 JUN 61
Test Lateral Pressure 6.50 kg/cm²

mm 1. 80.50 2. 80.56 Aver: 80.53
m Top 1. 35.96 2. 36.00 Aver: 35.98 Area Top A_T 10.167 sq.mm
entre 1. 35.88 2. 35.72 Aver: 35.80 Area Centre A_C 10.066 sq.mm
ottom 1. 35.40 2. 35.50 Aver: 35.45 Area Bottom A_B 9.870 sq.mm

Volume 80.85 cc

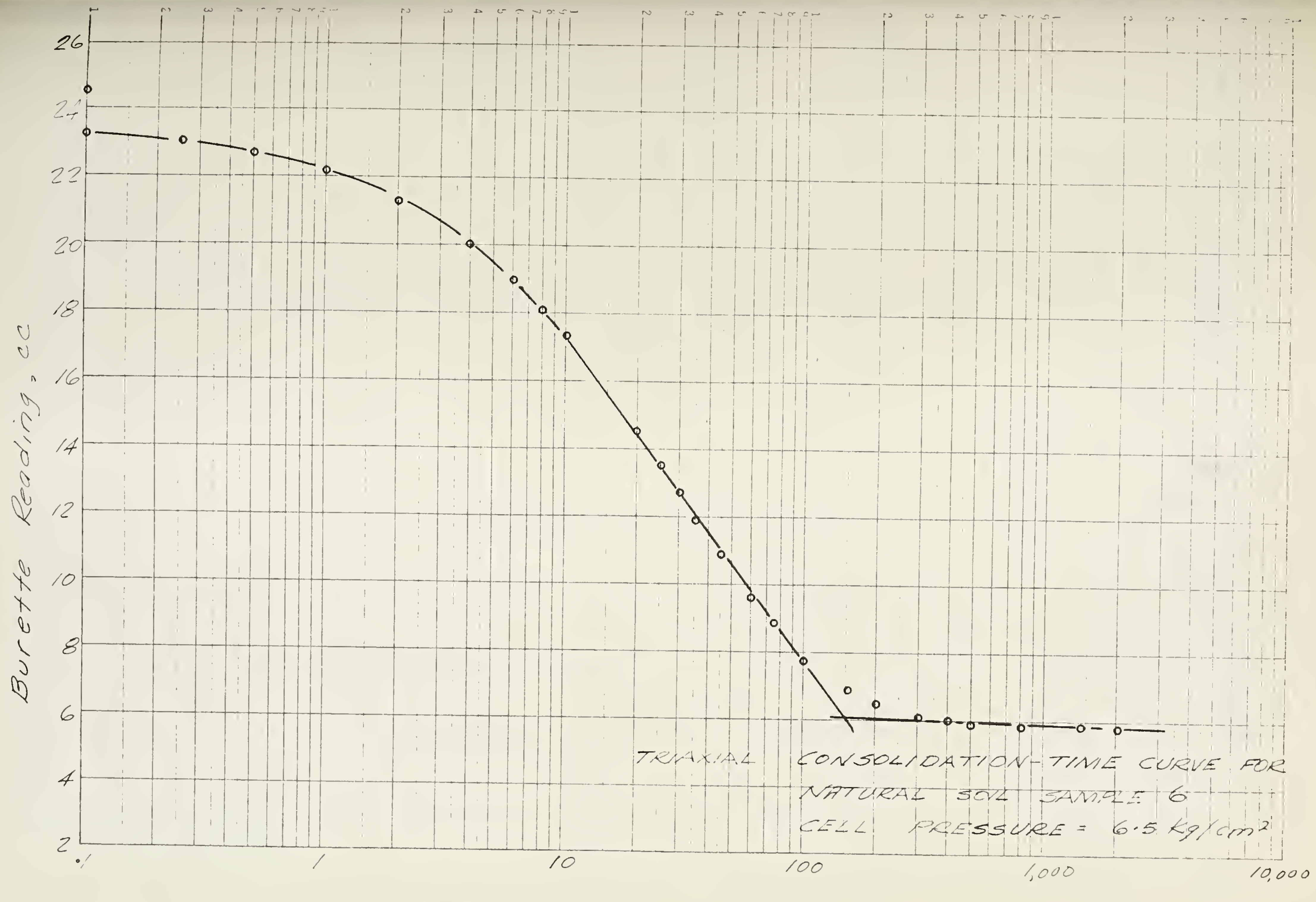
Average X–Sect Area
$= \frac{1}{4}(A_T + 2A_C + A_B)$
= 10.04

IDATION DATA

	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
50	0	24.50	
	~~25~~ .1	23.20	
	~~.5~~ .25	23.0	
	~~1.0~~ .50	22.70	
	1.0	22.15	
	2	21.30	
	4	20.05	
	6	19.0	
	8	18.15	
	10	17.35	
	15	15.75	
	20	14.55	
	25	13.50	
	30	12.65	
	35	11.90	
	45	10.80	
	60	9.60	
	75	8.75	
	100	7.80	
	152	6.85	

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
14 JUN	200	6.45	
	300	6.15	
	400	6.00	
	500	5.90	
14 JUN 2220	810	5.75	
15 JUN 0835	1425	5.7~~0~~5	
✓ 1810	2000	5.60	18.90



TRIAXIAL CONSOLIDATION-TIME CURVE FOR
NATURAL SOIL SAMPLE 6
CELL PRESSURE = 6·5 kg/cm²

PLATE F13

Sheet 1 F33

UNIVERSITY OF ALBERTA

epartment of Civil Engineering

Soil Mechanics Laboratory

COMPRESSION TEST ON COHESIVE SOIL

ilure

) = ________ σ_1= ________

%

Project THESIS

Hole No. Natural

Depth ________ Sample 6

Engineer ST

Technician DW

Date of Test 16 JUN 64

Test Lateral Pressure σ_3 6.50 Kg/cm²

Back Pressure 2.0 Kg/cm²

Remarks

Area Correction Factor 1.20

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1-\sigma_3}$ %
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
0	10.04	0	—	0	-0.12					
1	10.04	25	.0520	0.16	-.13				.01	
2	10.04	50	.0506	0.30	+.00				.025	
4	10.045	91	.0489	0.53	+.03	7.00	6.47	1.08	.05	5.7
8	10.05	167	.04685	0.93	+.20	7.23	6.30	1.15	0.10	21.5
16	10.06	269	.04536	1.46	+.48	7.48	6.02	1.24	0.20	32.9
24	10.07	334	.04473	1.78	.72	7.56	5.78	1.31	0.30	40.4
32.5	10.08	386	.0443	2.04	.89	7.65	5.61	1.36	0.40	43.6
40.5	10.09	426	.04402	2.23	1.05	7.68	5.45	1.41	0.50	47.1
48.5	10.10	459	.04384	2.39	1.16	7.73	5.34	1.45	.60	48.5
56.5	10.11	484	.04370	2.51	1.29	7.72	5.21	1.48	.70	51.4
64.5	10.12	506	.04356	2.61	1.42	7.69	5.08	1.51	.80	54.4
72.5	10.13	524	.04347	2.70	1.52	7.68	4.98	1.54	.90	56.3
80.5	10.14	541	.04340	2.78	1.58	7.70	4.92	1.57	1.00	56.8
88.5	10.15	555	.04332	2.84	1.67	7.67	4.83	1.59	1.10	58.8
96.5	10.16	567	.04328	2.90	1.72	7.68	4.78	1.61	1.20	59.3
104.5	10.17	579	.04322	2.95	1.80	7.65	4.70	1.63	1.30	61.0
112.5	10.18	591	.04318	3.01	1.86	7.65	4.64	1.65	1.40	61.7
121	10.19	602	.04313	3.06	1.95	7.61	4.55	1.67	1.50	63.7
129	10.205	613	.04308	3.11	1.99	7.62	4.51	1.69	1.60	63.9
137	10.215	622	.04305	3.15	2.03	7.62	4.47	1.70	1.70	64.4
145	10.225	630	.04301	3.18	2.10	7.58	4.40	1.72	1.80	66.0
153	10.235	638	.04300	3.22	2.15	7.57	4.35	1.74	1.90	66.7
161	10.245	647	.04295	3.25	2.18	7.57	4.32	1.75	2.00	67.1
177	10.27	661	.04290	3.31	2.28	7.53	4.22	1.78	2.20	68.9
193	10.285	675	.04285	3.37	2.35	7.52	4.15	1.81	2.40	69.7
209	10.305	687	.04280	3.42	2.42	7.50	4.08	1.84	2.60	70.8
225.5	10.33	700	.04277	3.48	2.50	7.48	4.00	1.87	2.80	71.8
241.5	10.35	712	.04271	3.53	2.57	7.46	3.93	1.90	3.00	72.8

UNIVERSITY OF ALBERTA

Department of Civil Engineering
Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth ______ Sample 6

Area Correction Factor 1.20

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1 - \sigma_3}$
257.5	10.37	722	.04268	3.57	2.63	7.44	3.87	1.92	3.20	73.6
274	10.39	733	.04263	3.61	2.65	7.46	3.85	1.94	3.40	73.4
290	10.415	743	.04260	3.65	2.68	7.47	3.82	1.96	3.60	73.4
306	10.435	754	.04257	3.69	2.71	7.48	3.79	1.97	3.80	73.4
322	10.455	761	.04255	3.72	2.74	7.48	3.76	1.99	4.00	73.7
338	10.48	771	.04250	3.75	2.88	7.37	3.62	2.04	4.20	76.8
354	10.50	779	.04248	3.78	2.94	7.34	3.56	2.06	4.40	77.8
370	10.525	787	.04245	3.81	2.96	7.35	3.54	2.08	4.60	77.7
386.5	10.55	796	.04240	3.84	3.00	7.34	3.50	2.10	4.80	78.1
402.5	10.57	804.	.04239	3.87	3.02	7.35	3.48	2.11	5.00	78.0
418.5	10.59	811	.04236	3.89	3.06	7.33	3.44	2.13	5.20	78.6
435	10.615	817	.04233	3.91	3.07	7.34	3.43	2.14	5.40	78.5
451	10.64	823	.04231	3.93	3.09	7.34	3.41	2.15	5.60	78.6
467	10.66	830	.04230	3.95	3.12	7.33	3.38	2.17	5.80	79.0
483	10.68	836	.04228	3.97	3.14	7.33	3.36	2.18	6.00	79.1
499	10.70	843	.04226	4.00	3.17	7.33	3.33	2.20	6.20	79.3
515.5	10.73	848	.04223	4.00	3.19	7.31	3.31	2.21	6.40	79.8
547	10.77	859	.04220	4.04	3.23	7.31	3.27	2.24	6.80	80.0
563.5	10.795	864	.04218	4.05	3.25	7.30	3.25	2.25	7.00	80.2
604	10.855	875	.04214	4.08	3.30	7.28	3.20	2.28	7.50	80.8
620	10.88	879	.04213	4.08	3.31	7.27	3.19	2.28	7.70	81.1
749	11.075	905	.04203	4.12	3.44	7.18	3.06	2.35	9.30	83.5
765	11.10	905	.04203	4.11	3.42	7.19	3.08	2.33	9.50	83.2
785	11.13	905	.04203	4.10	3.44	7.16	3.06	2.34	9.75	83.9
805	11.16	905	.04203	4.09	3.46	7.13	3.40	2.35	10.0	84.6 ←
845.5	11.22	906	.04202	4.07	3.46	7.11	3.04	2.34	10.50	85.0
865.5	11.25	905.5	.04203	4.06	3.45	7.11	3.05	2.33	10.75	85.0
885.5	11.285	905.5	.04203	4.05	3.47	7.08	3.03	2.34	11.00	85.7

ontent – Initial 52.54

Final 33.25%

aturation – Initial 97.3

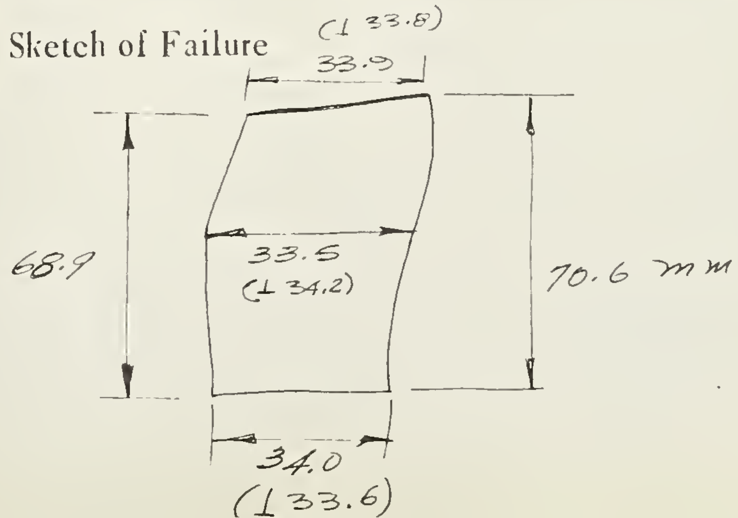
Final 97.1

– Initial 1.502

Final 0.952

ure Reaction 85%

Sketch of Failure



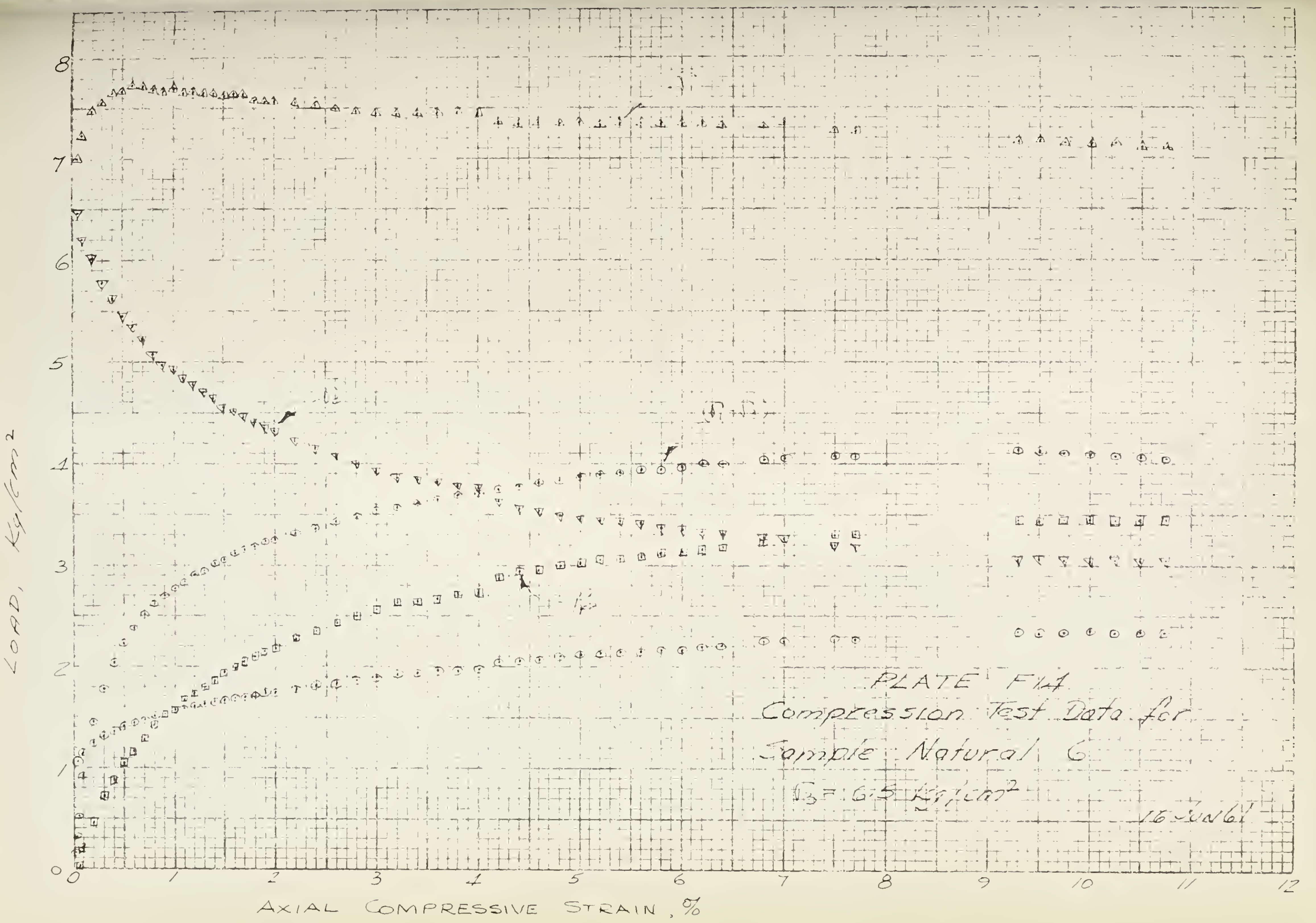


PLATE F14
Compression Test Data for
Sample Natural 6
$\sigma_3 = 6.5$ Kg/cm²
16-Jun-61

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Natural
Depth sample 7
Engineer ST
Technician
Date of Set-up 27 Oct 61
Test Lateral Pressure 7.5 kg/cm²

e Description

5 internal Wicks
Weight 134.912 gms

mm	1. 80.54	2. 80.52	Aver: 80.53	Area Top A_T 9.759 sq.$\overset{c}{m}$m
m Top	1. 35.28	2. 35.22	Aver: 35.25	Area Centre A_C 9.926 sq.$\overset{c}{m}$m
entre	1. 35.56	2. 35.60	Aver: 35.55	Area Bottom A_B 10.015 sq.$\overset{c}{m}$m
ottom	1. 35.70	2. 35.72	Aver: 35.71	Average X-Sect Area

Volume 79.78 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

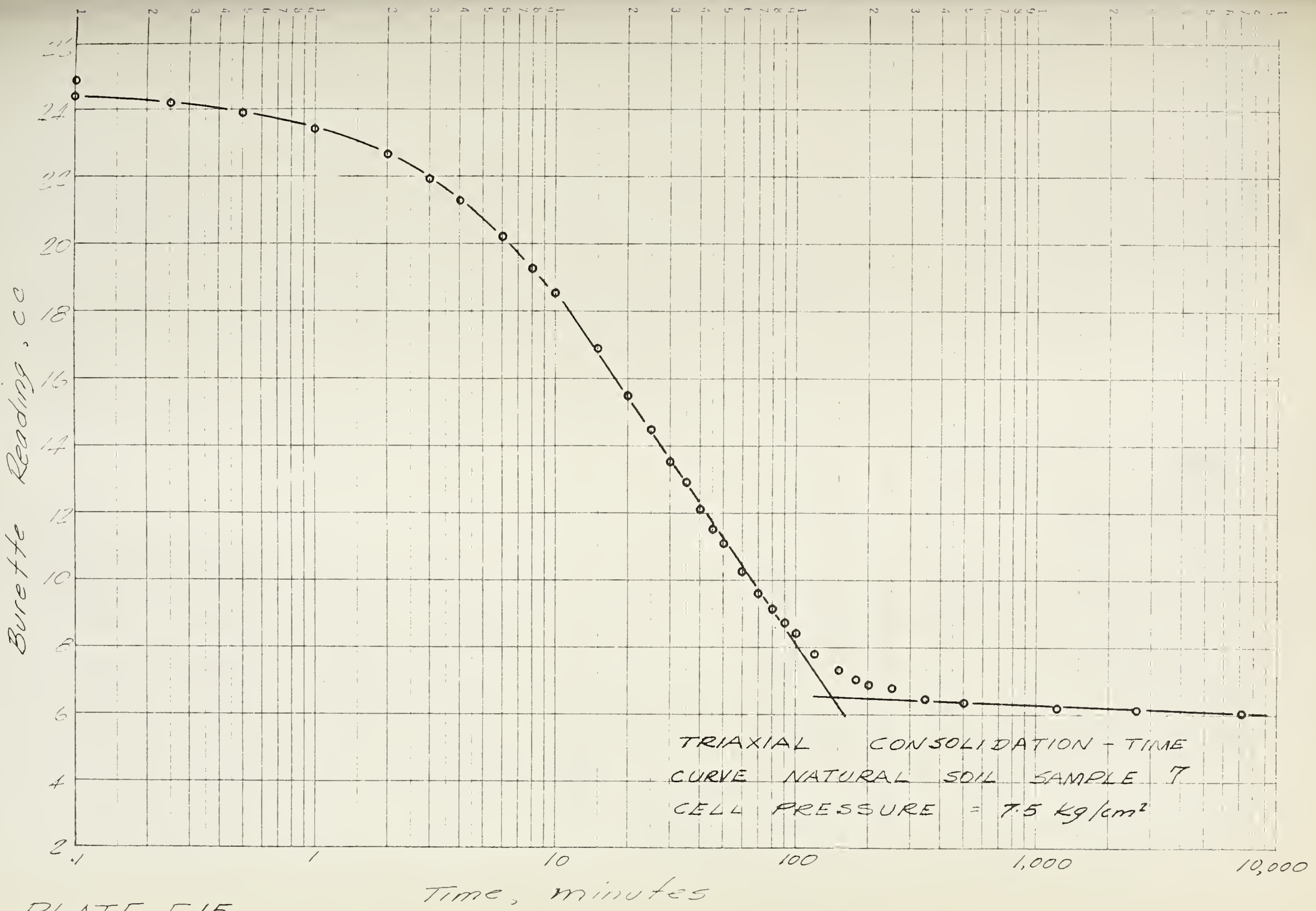
$=$ 9.91 cm²

IDATION DATA

	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
200	0	24.80	
	.1	24.45	
	.25	24.20	
	.50	23.95	
	1	23.40	
	2	22.65	
	3	21.90	
	4	21.30	
	6	20.20	
	8	19.30	
	10	18.50	
	15	16.85	
	20	15.50	
	25	14.40	
	30	13.52	
	35	12.80	
	40	12.13	
	45	11.55	
	50	11.10	
	60	10.28	

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
	70	9.63	
	80	9.10	
	90	8.70	
	100	8.40	
	120	7.85	
	150	7.35	
	180	7.05	
	200	6.90	
	250	6.70	
1745	345	6.42	
2020	500	6.30	
28 Oct 0815	1225	6.15	
30 " 0815	2665	6.10	
1 Nov 1645	6055	6.03	18.77
			(K= 1.20)



TRIAXIAL CONSOLIDATION - TIME CURVE NATURAL SOIL SAMPLE 7

CELL PRESSURE = 7.5 kg/cm²

PLATE F15

Sheet 1

F38

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

COMPRESSION TEST ON COHESIVE SOIL

ailure

3) = ____________ σ_1= ____________

____

____

____

____________ %

Project THESIS

Hole No. Natural

Depth ____________ Sample 7

Engineer ST

Technician ____________

Date of Test 2 Nov 61

Test Lateral Pressure σ_3 7.5 kg/cm²

Back Pressure 2

Remarks ____________

Rate of Strain 362.20 min per 10mm

Proving Ring N° 3-1498

Area Correction Factor 1.20

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\bar{A}$
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		$\frac{P_p}{\sigma_1-\sigma_3}$
0	9.91	0	0							
1	✓	20	.0525	.127	0				.012	
2.5	9.91	40	.0511	.248	+.03				.031	
3.5	9.915	60	.0501	.364					.043	
8	9.92	180	.04660	1.01	+.26	8.25	7.24	1.140	.1	25.7
16	9.93	255	.04553	1.40	.65	8.25	6.85	1.204	.2	46.4
24	9.94	356	.04456	1.91	.97	8.44	6.53	1.292	.3	50.8
32	9.95	425	.04404	2.26	1.15	8.61	6.35	1.356	.4	50.9
40	9.96	473	.04375	2.49	1.37	8.62	6.13	1.406	.5	55.0
48	9.97	510	.04355	2.67	1.53	8.64	5.97	1.447	.6	57.3
56.5	9.98	540	.04340	2.82	1.68	8.64	5.82	1.484	.7	59.6
64.5	9.99	563	.04330	2.93	1.81	8.62	5.69	1.515	.8	61.8
72.5	10.00	582	.04320	3.02	1.92	8.60	5.58	1.541	.9	63.0
80.5	10.01	599	.04313	3.10	2.01	8.59	5.49	1.565	1.0	64.8
89	10.02	615	.04308	3.17	2.13	8.54	5.37	1.590	1.1	67.1
97	10.03	629	.04302	3.24	2.20	8.54	5.30	1.611	1.2	67.9
105	10.04	641	.04299	3.29	2.30	8.49	5.20	1.633	1.3	69.9
113	10.05	650	.04294	3.33	2.39	8.44	5.11	1.652	1.4	71.8
121	10.06	659	.04290	3.37	2.44	8.43	5.06	1.666	1.5	72.4
129	10.07	666	.04289	3.40	2.52	8.38	4.98	1.683	1.6	74.1
137	10.08	675	.04286	3.44	2.54	8.40	4.96	1.694	1.7	73.8
145	10.09	684	.04282	3.48	2.63	8.35	4.87	1.715	1.8	75.6
153	10.10	691	.04280	3.51	2.68	8.33	4.82	1.728	1.9	76.4
161	10.11	698	.04277	3.54	2.73	8.31	4.77	1.742	2.0	77.1
169	10.12	704	.04274	3.57	2.78	8.29	4.72	1.756	2.1	77.9
177	10.13	~~42~~ 711	.04272	3.60	2.82	8.28	4.68	1.769	2.2	78.6
193	10.15	721	.04269	3.64	2.94	8.20	4.56	1.798	2.4	80.8
209.5	10.175	733	.04264	3.69	2.98	8.21	4.52	1.816	2.6	80.8
225.5	10.195	743	.04260	3.73	3.06	8.17	4.46	1.832	2.8	82.0

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Natural

Depth

Sample 7

Area Correction Factor 1.20

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_f	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\bar{\sigma}_1 / \bar{\sigma}_3$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1-\sigma_3}$
241.5	10.215	751	.04258	3.76	3.16	8.10	4.34	1.866	3.0	84.0
282	10.27	771	.04251	3.83	3.28	8.05	4.22	1.908	3.5	85.6
322	10.32	788	.04245	3.89	3.42	7.97	4.08	1.953	4.0	87.9
362	10.38	802	.04240	3.93	3.54	7.89	3.96	1.992	4.5	90.1
402.5	10.43	812	.04236	3.96	3.66	7.80	3.84	2.031	5.0	92.4
443	10.485	819	.04233	3.71	3.71	7.76	3.79	2.047	5.5	93.5
483	10.54	824	.04231	3.97	3.76	7.71	3.74	2.061	6.0	94.7
523.5	10.60	827 ~~425~~	.04230	3.96	3.83	7.63	3.67	2.079	6.5	96.7
564	10.655	829	.04230	3.95	3.87	7.58	3.63	2.088	7.0	98.0
604	10.71	827	.04230	3.93	3.91	7.52	3.59	2.095	7.5	99.4
44	10.77	821	.04230	3.91	3.94	7.47	3.56	2.098	8.0	100.8
684.5	10.83	827	.04230	3.88 ★	3.97	7.41	3.53	2.099	8.5	103.1
725	10.89	833	.04229	3.88	4.02	7.36	3.48	2.115	9.0	103.6
765	10.95	844	.04225	3.91	4.04	7.37	3.46	2.130	9.5	103.3
805	11.01	852	.04222	3.92	4.06	7.36	3.44	2.140	10.0	103.6
845.5	11.07	856	.04220	3.92	4.08	7.34	3.42	2.146	10.5	104.1
886	11.135	859	.04219	3.91	4.08	7.33	3.42	2.143	11.0	104.3
926	11.20	861	.04219	3.89	4.09	7.30	3.41	2.141	11.5	105.1

★ slight flat "s" shaped distortion becoming visible

ontent – Initial 54.0

Final 32.67

aturation – Initial 97.9 %

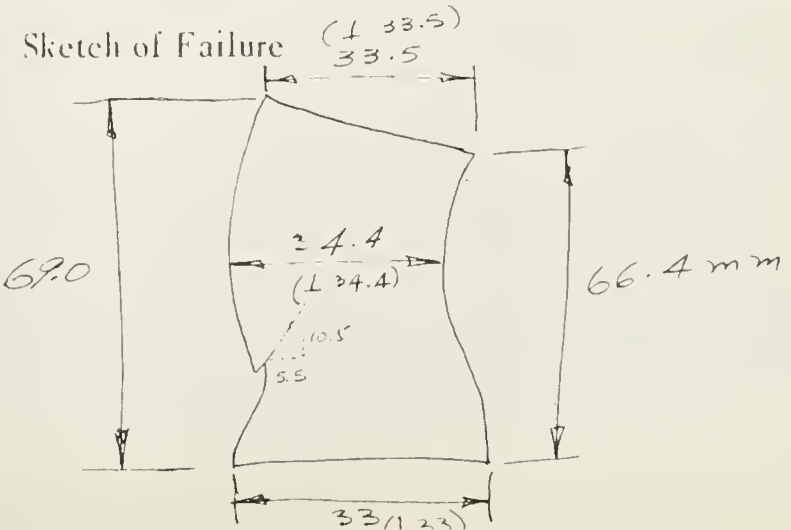
Final 97.2

– Initial 1.530

Final 0.934

re Reaction 65%

Sketch of Failure



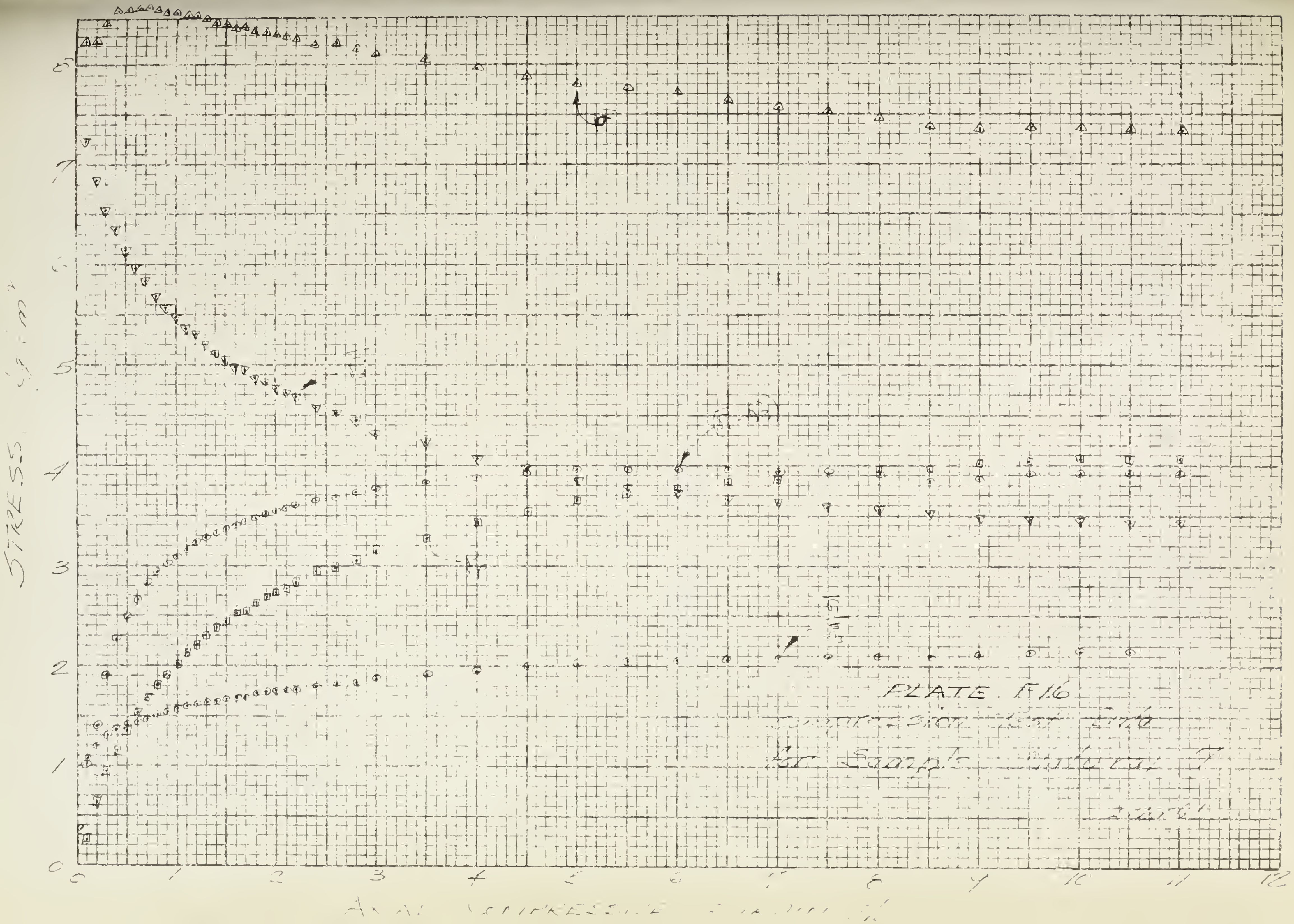


PLATE F16

Compression Test [illegible]

[illegible] Sample [illegible]

APPENDIX G

DATA SHEETS FOR TESTS ON POTASSIUM MODIFIED SOIL

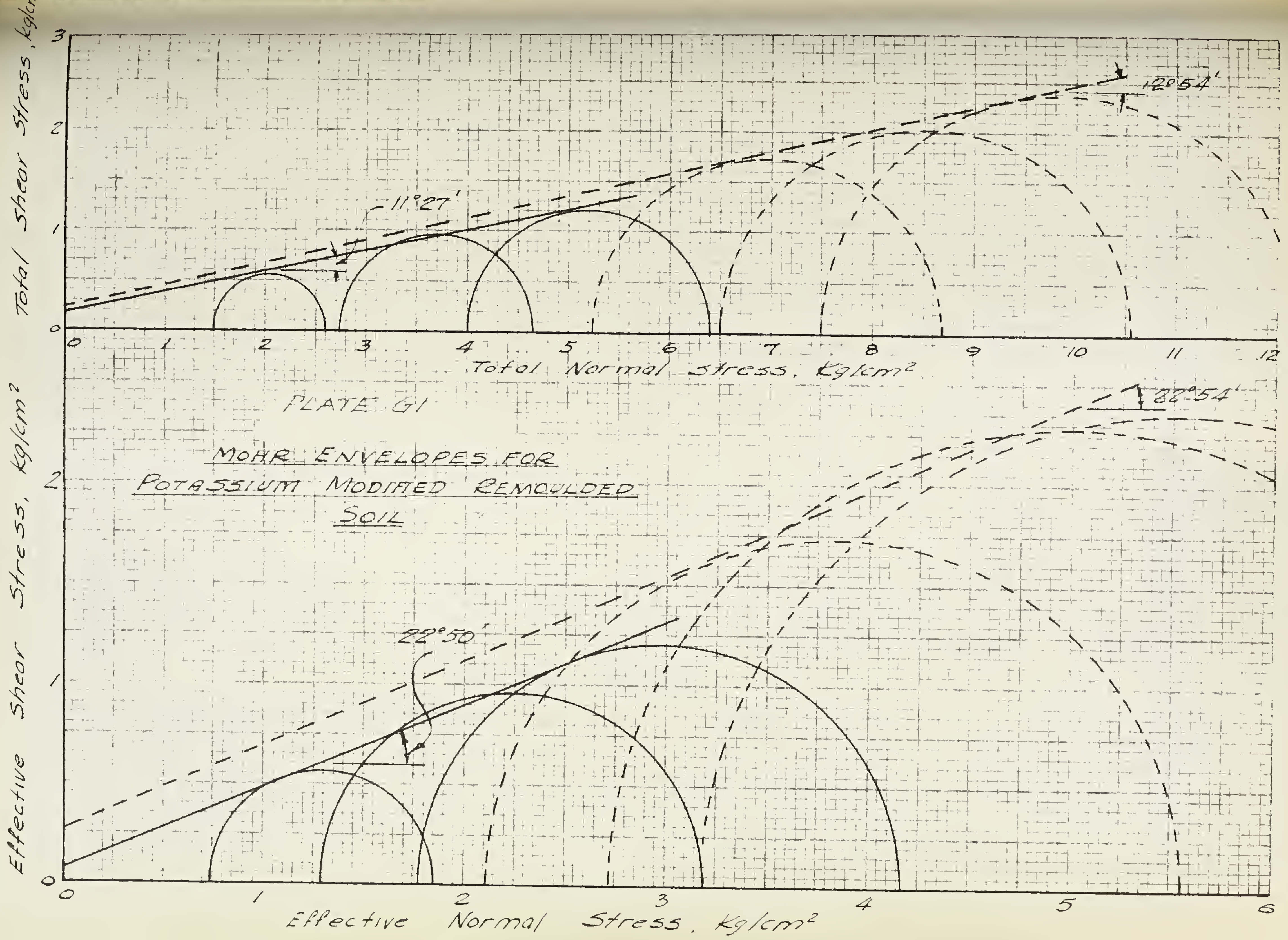


PLATE G1

MOHR ENVELOPES FOR
POTASSIUM MODIFIED REMOULDED
SOIL

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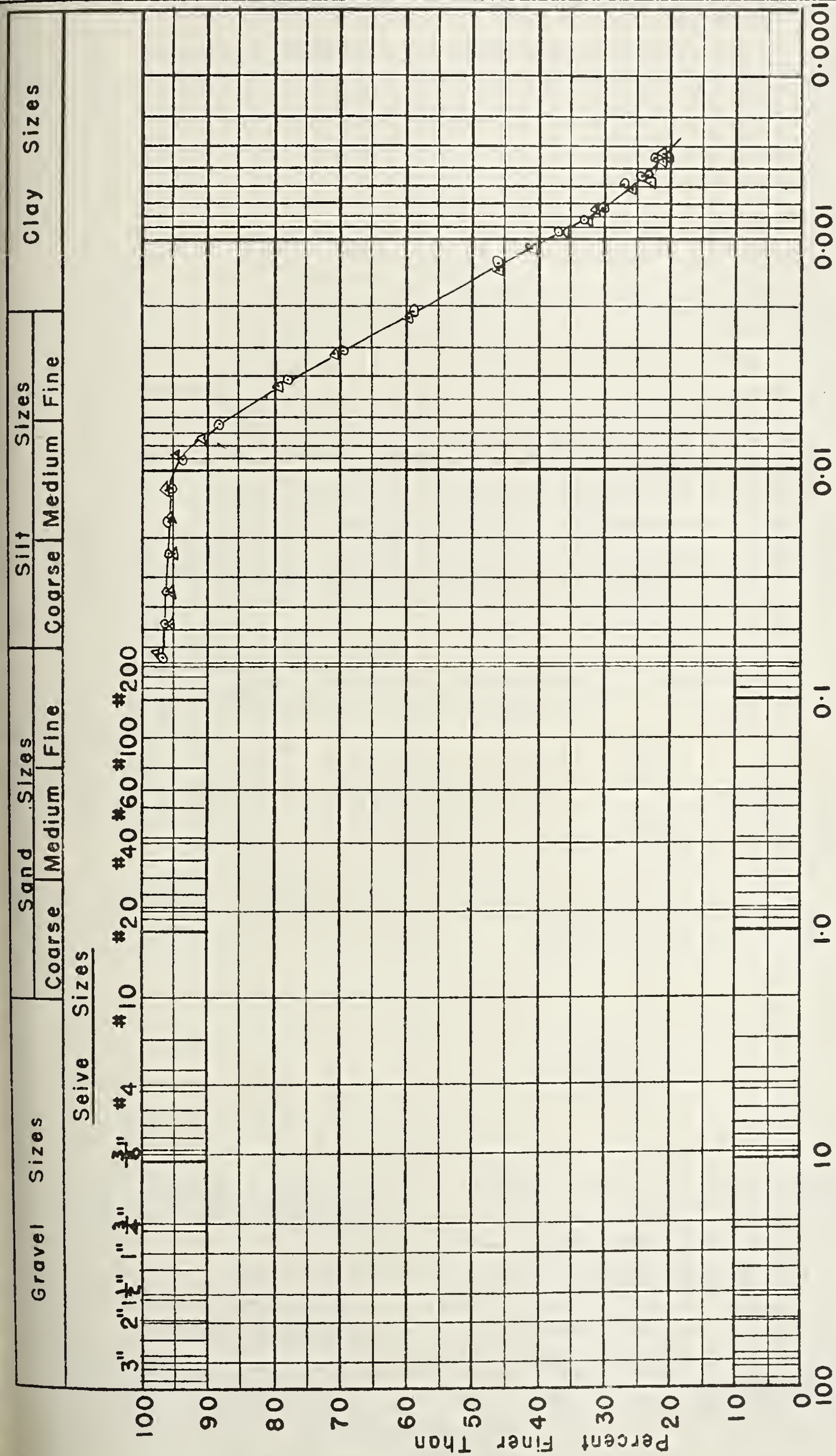
SOIL MECHANICS LABORATORY

GRAIN SIZE CURVE

PROJECT	THESIS		
SITE			
SAMPLE	Potassium Gp 1		
LOCATION			
HOLE		DEPTH	
TECHNICIAN	DW	DATE	14 JUN 61



D₁₀ = ____ mm.
D₆₀ = ____ mm.
Cu ____

Legend:
Sample 1 ⊙
Sample 2 △

Remarks: From work of Hamilton:
W_L = 69.5%
W_p = 32.6%
I_p = 36.9%
Compressive Index 0.50

Note: M.I.T. Grain Size Scale

PLATE G2

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SOIL MECHANICS LABORATORY

HYDROMETER TEST

PROJECT THESIS
SITE
SAMPLE Potassium #2
LOCATION
HOLE DEPTH
TECHNICIAN DW DATE 2JUN61

Date	Temp. °C	Time	Elapsed Time	R'_h	$R_h = R'_h + c_m$	D m.m.	$R_h + m_t - c_d$	W.%	W % Basis Orig. Sample	Remarks
2Jun61		1147	.5	19.7	20.2	.0675	20.8	96.8		
			1 min	19.5	20.0	.0480	20.6	95.9		
			2	19.4	19.9	.0340	20.5	95.4		
			4	19.3	19.8	.0240	20.4	95.0		
	24.4		8	19.6	20.1	.0170	20.7	96.3		
			15	19.7	20.2	.0125	20.8	96.8		
	24.3		30	19.2	19.7	.0088	20.3	94.5		
	24.1		46	18.4	18.9	.0073	19.4	90.3		
	23.5		2h 20m	16.1	16.6	.0044	17.0	79.1		
	23.3		4:10	14.3	14.8	.0032	15.2	70.7		
2JUN	23.3	2130	9:43	11.9	12.4	.0022	12.8	59.6		
3JUN	23.3	1330	25:43	8.9	9.4	.0014	9.8	45.6		
3 ✓	23.3	2330	35:43	8.0 5.7	8.5 6.2	00115 .00073	8.9 6.6	41.4 30.7		
4 ✓	23.2	2130	57:43	7.0 4.8	7.5 5.3	00093 00060	7.8 5.6	36.3 26.1		
5 ✓	24.4	1347	74:00	6.1 3.9	6.6 4.4	00082 00054	7.2 5.0	33.5 23.3		
6 ✓	24.6	0940	93:53	5.4 3.7	5.9 4.2	00073 .00043	6.5 4.8	30.3 22.3		
6 ✓	25.3	1640	100:53	3.4 ~~2.9~~	3.9	.00046	4.7	21.9		
7 ✓	24.9	0840	116:53	3.3	3.8	.00042	~~4~~5.5	20.9		

Hydrometer No.s 9857R and 50700R Graduate No.

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s - 1} (R_h + m_t - c_d) = 4.65 \ (R_h + m_t - c_d)$

Meniscus correction = c_m = 0.5 and 0.5 respectively

Dispersing agent used Calgon Amount 10 ml

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = 0.3 and 0.3 respectively

Specific Gravity of Solids = G_s 2.74

Description of Sample

Method of Preparation

Remarks

Initial Moisture Content

Container No.
Wt. Sample Wet + Tare
Wt. Sample Dry + Tare
Wt. Water
Tare Container
Wt. of Dry Soil
Initial Moisture w %

Dry Weight of Sample

Container No.
Wt. Sample (~~Wet~~/Dry) + Tare 1017.8
Tare #2 983.9
Wt. (~~Wet~~/Dry) Soil 33.9
Dry Weight from Initial

Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist.}\%}$ =

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HYDROMETER TEST

PROJECT THESIS
SITE
SAMPLE Potassium (GPI) #1
LOCATION
HOLE DEPTH
TECHNICIAN DW DATE 2 JUN 61

Date	Temp. °C	Time	Elapsed Time	R_h'	R_h $=R_h'+c_m$	D m.m.	$R_h+m_t-c_d$	W.%	W % Basis Orig. Sample	Remarks
2 Jun 61		1130	.5	19.9	20.4	.067	21.0	97.2		
		1131	1 MIN	19.8	20.3	.047	20.9	96.8		
			2	19.7	20.2	.034	20.8	96.3		
			4	19.6	20.1	.024	20.7	95.8		
	24.6		8	19.6	20.1	.017	20.7	95.8		
			15	19.5	20.0	.0125	20.6	95.4		
	24.5		30	19.1	19.6	.0090	20.2	93.5		
	24.2		60	18.0	18.5	.0063	19.0	88.0		
	23.5		2h:35m	16.0	16.5	.0041	16.9	78.2		
	23.3		4:26	14.2	14.7	.0031	15.1	69.9		
2 Jun	23.3	2130	10:00	11.9	12.4	.00215	12.8	59.3		
3 ✓	23.3	1330	26:00	9.0	9.5	.00136	9.9	45.8		
3 ✓	23.3	2330	36:00	8.0 5.7	8.5 6.2	.00117 .00073	8.9 6.6	41.2 30.6		
4 ✓	23.3	2130	58:00	7.0 4.9	7.5 5.4	.00093 .00059	7.9 5.8	36.6 26.9		
5 ✓	24.0	1345	74:15	6.2 4.2	6.7 4.7	.0008 .00052	7.2 5.2	33.3 24.1		
6 ✓	24.5	0935	94:05	5.3 3.8	5.8 4.3	.00072 .00052	6.4 4.9	29.6 22.7		
6 ✓	25.4	1640	101:10	3.6	4.1	00045	4.9	22.7		
7 ✓	24.9	0840	117:10	3.2	3.7	.00044	4.5	20.4		

Hydrometer No.s 9857R and 50700 R Graduate No.

$W\% = \frac{100}{W_s}\cdot\frac{S_s}{S_s-1}(R_h+m_t-c_d)' =$ 4.63 $(R_h+m_t-c_d)$

Meniscus correction = c_m = 0.5 and 0.5 respectively

Dispersing agent used Calgon Amount 10 ml

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = 0.3 and 0.3 respectively

Specific Gravity of Solids = G_s 2.74

Description of Sample

Method of Preparation

Remarks

Initial Moisture Content

Container No.
Wt. Sample Wet + Tare
Wt. Sample Dry + Tare
Wt. Water
Tare Container
Wt. of Dry Soil
Initial Moisture w %

Dry Weight of Sample

Container No.
Wt. Sample (Wet/Dry) + Tare 1151.9
Tare 1117.9
Wt. (Wet/Dry) Soil 34.0
Dry Weight from Initial Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist.}\%}$ =

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SPECIFIC GRAVITY

PROJECT THESIS
SITE
SAMPLE Potassium
LOCATION
HOLE DEPTH
TECHNICIAN DW & ST DATE 14 July 61

Sample No.	1 ★	2 ‡	3 ‡	4 ×	5 +
Flask No.	101	100	100	101	100
Method of Air Removal	Vacuum	Vacuum	Vacuum	Vacuum	Vacuum
W_{b+w+s}	688.46	690.53	693.38	680.26	693.38
Temperature T	28.8°C	24.6°C	30.3°C	23.3°C	30.3°C
W_{b+w}	666.82	669.48	668.70	667.52	668.70
Evaporating Dish No.					
Wt. Sample Dry + Dish	224.60	220.38	228.26	204.95	228.26
Tare Dish	190.50	186.94	188.82	184.80	188.82
W_s	34.10	33.44	39.44	20.15	39.44
G_s	2.74	2.70	2.67	2.72	2.67

W_{b+w+s} = Weight of flask + water + sample at T°.
W_{b+w} = Weight of flask + water at T° (flask calibration curve).
W_s = Weight of dry soil
G_s = Specific gravity of soil particles $= \dfrac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.			Sample No.		
Container No.			Container No.		
Wt. Sample Wet + Tare			Wt. Test Sample Wet + Tare		
Wt. Sample Dry + Tare			Tare Container		
Wt. Water			Wt. Test Sample Wet		
Tare Container			W_s		
Wt. of Dry Soil					
Moisture Content w %					

Description of Sample:

Remarks: ★ tested 7 JUN 61
‡ " 21 JUN 61
× " 14 July 61
+ " 13 July 61

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Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Potassium Gp 1

Depth sample 1

Engineer ST

Technician DW

Date of Set-up 19 May 61

Test Lateral Pressure 1·5 kg/cm²

mple Description Average Cation Analysis:
18·6%, Na 0·5%, K 75·8%, Mg 5·1%
tion Exchange Capacity 29·0 me/100 gm Soil
alt Content 31·8 me/100 gm soil
w 5 internal wool wicks.

itial Weight 135.567 gms

gth, mm	1. 80·06	2. 80·08	Aver: 80·07	Area Top	A_T 9.781	sq.mm
. mm Top	1. 35·20	2. 35·38	Aver: 35·29	Area Centre	A_C 9.976	sq.mm
Centre	1. 35·60	2. 35·68	Aver: 35·64	Area Bottom	A_B 10·060	sq.mm
Bottom	1. 35.80	2. 35·78	Aver: 35·79			

nal Volume 79.65 cc

Average X-Sect Area

$= \frac{1}{4}(A_T + 2A_C + A_B)$

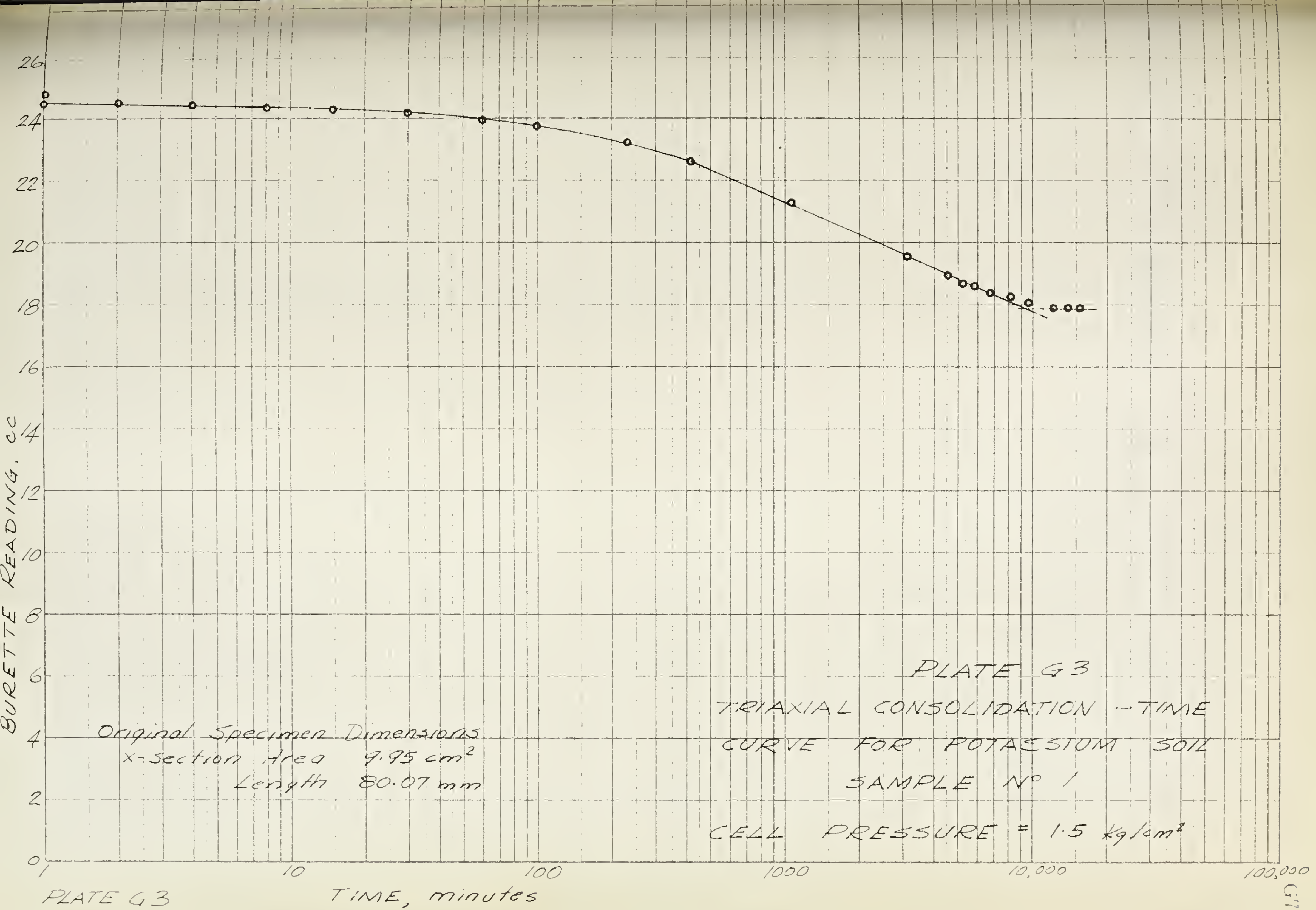
= 9.95

SOLIDATION DATA

e e	Δt min.	Burette Rdg. c.c.	ΔV c.c.
16/1555	0	24·80	
	·1	24·60	
	·25	24·55	
	·5	24·53	
	1	24·50	
	2	24·50	
	4	24·45	
	8	24·40	
	15	24·30	
	30	24.15	
	60	23.95	
	100	23·75	
1945	230	23·20	
2245	410	22·65	
0945	1070	21·30	
2045	3170	19.55	
2005	4570	18·95	
0840	5320	18·70	
700	5820	18·60	
0845	6765	18·40	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
25 May 0845	8205	18·20	
26 ✓ 0845	9645	18·05	
28 ✓ 0050	12,050	17·90	
29 ✓ 0835	13,955	17·90	
30 ✓ 0900	15,420	17·90	



Original Specimen Dimensions
X-Section Area 9.95 cm²
Length 80.07 mm

PLATE G3

TRIAXIAL CONSOLIDATION – TIME
CURVE FOR POTASSIUM SOIL
SAMPLE No 1

CELL PRESSURE = 1.5 kg/cm²

PLATE G3

Sheet 1 G8

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Potassium # Gp 1

Depth Sample 1

Engineer ST

Technician DW

Date of Test 31st May 61

Test Lateral Pressure σ_3 1.5 kg/cm²

Back Pressure 2.0 ✓

Remarks Proving Ring 3-1498

Rate of Strain 600 min/10 mm

Area Correction Factor K = 1.06

Failure

σ_3) = 1.10 σ_1 = 2.60

0.77 kg/cm²

1.83

0.73

7.25 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$ %
4	9.95	20	.0525	.11	+.10	1.51	1.40	1.08	.05	.91
5	9.95	35	.0514	.19	+.12	1.57	1.38	1.14	.06	.63
8	9.96	50	.0506	.27	.13	1.64	1.37	1.20	.10	.48
16	9.97	86	.0491	.45	.18	1.77	1.32	1.34	.20	40
24	9.98	108	.04833	.55	.22	1.83	1.28	1.43	.30	40
32	9.99	123	.0479	.63	.25	1.88	1.25	1.50	.40	39.7
40	10.00	133	.04762	.66	.28	1.88	1.22	1.54	.450	42.4
50	10.01	146	.0473	.73	.30	1.93	1.20	1.61	.625	41.1
56	10.02	151	.0472	.75	.32	1.93	1.18	1.64	.70	42.7
64	10.03	158	.0470	.78	.36	1.92	1.14	1.68	.80	46.2
72	10.04	164	.04691	.81	.37	1.94	1.13	1.72	.90	45.7
80	10.05	168	.0468	.83	.39	1.94	1.11	1.75	1.00	47.0
88	10.06	172	.04675	.85	.40	1.95	1.10	1.77	1.10	47.0
96	10.07	177	.04667	.87	.42	1.95	1.08	1.81	1.20	48.3
104	10.08	180	.0466	.88	.44	1.94	1.06	1.83	1.30	50.0
112	10.09	183	.04656	.90	.46	1.94	1.04	1.87	1.40	51.1
120	10.10	186	.0465	.91	.47	1.94	1.03	1.88	1.5	51.6
128	10.11	190	.04644	.93	.50	1.93	1.00	1.93	1.6	53.8
136	10.12	193	.0464	.94	.51	1.93	0.99	1.95	1.7	54.3
144	10.13	195	.04637	.95	.52	1.93	0.98	1.97	1.8	54.7
152	10.14	198	.04633	.96	.54	1.92	0.96	2.00	1.9	56.2
160	10.15	201	.04626	.97	.55	1.92	0.95	2.02	2.00	56.7
176	10.17	204	.04622	.98	.58	1.90	0.92	2.07	2.2	59.2
192	10.19	209	.04615	1.00	.64	1.86	.86	2.16	2.4	64.8
208	10.22	212	.04610	1.01	.64	1.86	.86	2.17	2.60	63.3
224	10.24	216	.04602	1.03	.66	1.87	.84	2.23	2.80	64.1
240	10.26	219	.04600	1.04	.69	1.85	.810	2.28	3.00	66.3
256	10.28	220	.04599	1.04	.70	1.84	.80	2.30	3.20	67.3
272	10.30	222	.04596	1.05	.72	1.83	.78	2.35	3.40	68.5

Sheet 2
G9

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project *THESIS*

Hole No. *Potassium*

Depth ______ Sample *1*

Area Correction Factor *K = 1.06*

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress $\bar\sigma_1$ Major	Effective Stress $\bar\sigma_3$ Minor	Stress Ratio $\frac{\bar\sigma_1}{\bar\sigma_3}$	Axial Comp. Strain %	$\bar A$ $\frac{P_p}{\bar\sigma_1 - \bar\sigma_3}$
296	10.33	324	.04593	1.06	.73	1.83	.77	2.38	3.70	68.8
304	10.34	225.5	.04590	1.06	.74	1.82	.76	2.39	3.80	69.8
320	10.36	227	.04588	1.07	.75	1.82	.75	2.43	4.00	70.1
340	10.39	229	.04585	1.07	.74	1.83	.76	2.41	4.25	69.2
360	10.42	231.5	.04582	1.08	.70	1.88	.80	2.35	4.50	64.8
380	10.45	232	.04581	1.08	.72	1.86	.78	2.38	4.75	66.7
400	10.48	235	.04577	1.09	.72	1.87	.78	2.40	5.00	66.1
420	10.50	236.5	.04575	1.09 ★	.73	1.87	.77	2.43	5.25	67.0
440	10.53	238	.04572	1.10	.74	1.86	.76	2.45	5.50	67.2
460	10.56	240	.04571	1.10	.74	1.86	.76	2.45	5.75	67.2
480	10.59	241	.04570	1.10	.73	1.87	.77	2.43	6.00	66.4
500	10.62	242	.04569	1.10	.75	1.85	.75	2.47	6.25	68.2
520	10.64	243	.04567	1.11	.76	1.85	.74	2.50	6.50	68.4
540	10.67	244	.04565	1.11	.74	1.87	.76	2.46	6.75	66.7
560	10.70	245	.04564	1.11	.75	1.86	.75	2.48	7.00	67.6
580	10.73	245	.04564	1.10	.77	1.83	.73	2.51	7.25	70.0
600	10.76	245.5	.04564	1.10	.77	1.83	.73	2.51	7.50	70.0

Sample showing slight signs of distortion

Content – Initial *53.22*

Final *45.62*

f Saturation – Initial *99.8*

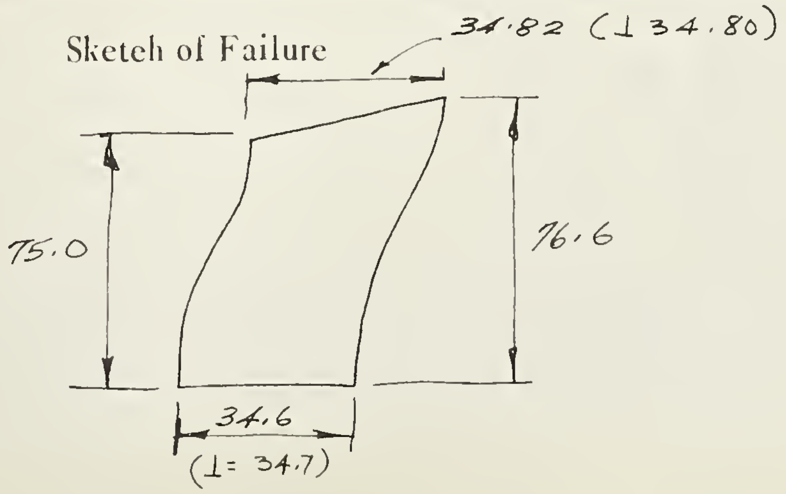
Final *—*

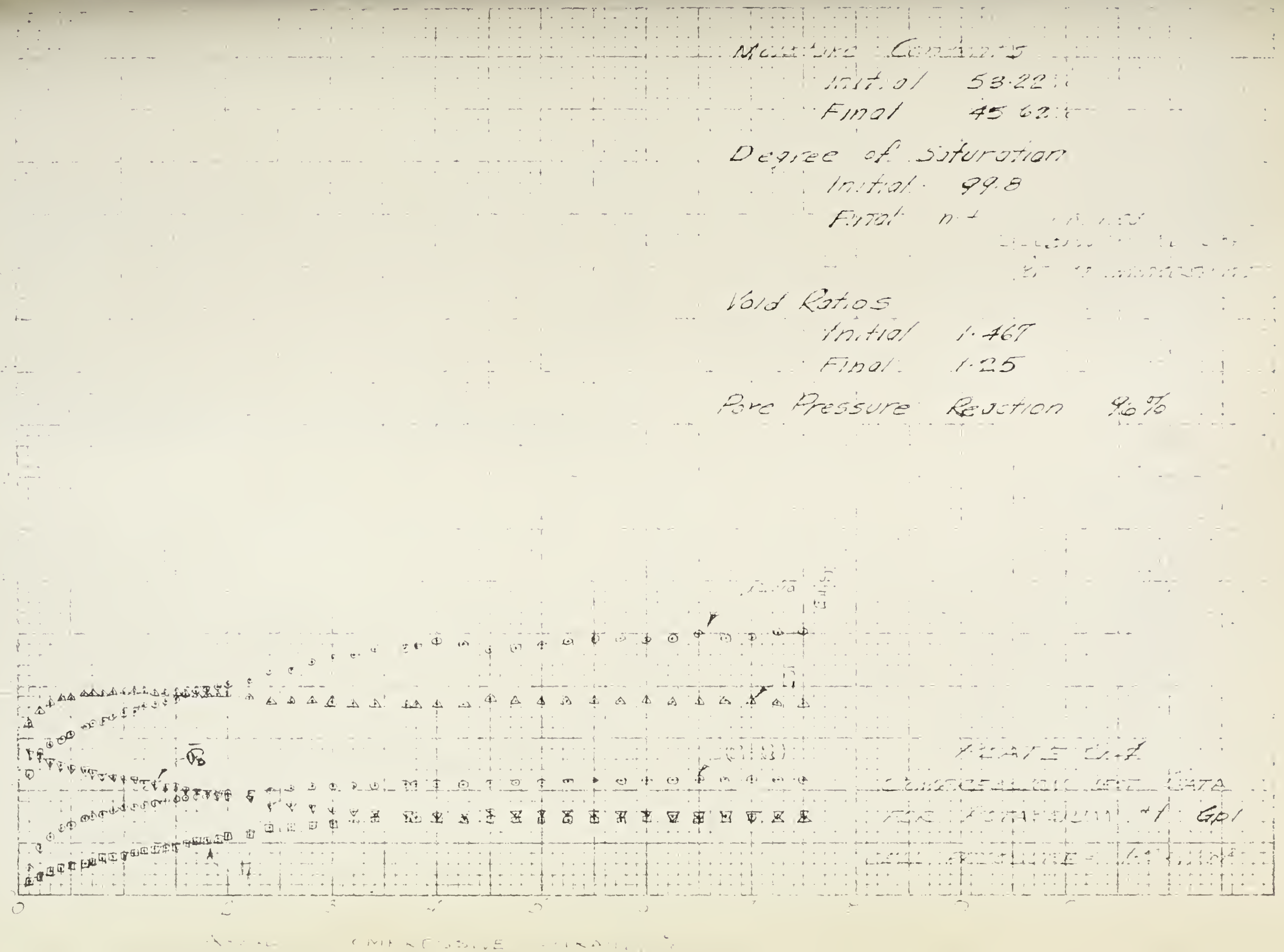
io – Initial *1.467*

Final *1.25*

ssure Reaction *96%*

Sketch of Failure




Moisture Contents
Initial 53.22
Final 45.62
Degree of Saturation
Initial 99.8
Final
Void Ratios
Initial 1.467
Final 1.25
Pore Pressure Reaction 96%
PLATE G.
TEST DATA
FOR No 1 Gp1
0
2
3
4
5
6
7
8

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Department of Civil Engineering
Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

ample Description CEC 29·0, SC 31·8
a 18·6, Na 0·5 K75·8 Mg 5·1

5 internal wicks

nitial Weight 134·346 gms

Project THESIS
Hole No. Potassium (Gp 1)
Depth sample 2
Engineer ST
Technician DW
Date of Set-up 19 May 61
Test Lateral Pressure 2·75

ngth, mm	1. 80·30	2. 80·36	Aver: 80·33	Area Top	A_T 10·055	sq.mm
m. mm Top	1. 35·82	2. 35·74	Aver. 35·78	Area Centre	A_C 9·865	sq.mm
Centre	1. 35·38	2. 35·50	Aver: 35·44	Area Bottom	A_B 9·787	sq.mm
Bottom	1. 35·30	2. 35·30	Aver: 35·30			

inal Volume 79·45 cc

Average X–Sect Area

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$=$ 9·89

SOLIDATION DATA

e e	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
1615	0	24·95	
	·25	24·85	
	·5	24·80	
	1	24·80	
	2	24·75	
	4	24·70	
	8	24·60	
	15	24·50	
	30	24·30	
	60	24·00	
	100	23·75	
945	210	23·25	
2245	390	22·65	
245	1050	21·20	
2045	3150	18·50	
2010	4555	17·30	
0840	5305	16·70	
1705	5810	16·35	
845	6750	15·80	
845	8190	15·20	

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
26 MAY 0845	9630	14·70	
28 ✓ 0050	12,035	14·05	
29 ✓ 0835	13,940	13·80	
30 ✓ 0900	15,405	13·65	
31 ✓ 0940	16,890	13·50	
1 JUN 0840	18,270	13·40	
2 ✓ 0840	19,710	13·30	
3 ✓ 1320	21,430	13·18	
5 ✓ 0830	24,020	13·08	
6 ✓ 0830	25,460	13·00	
7 ✓ 0830	26,900	13·00	11·95
			K=1·116

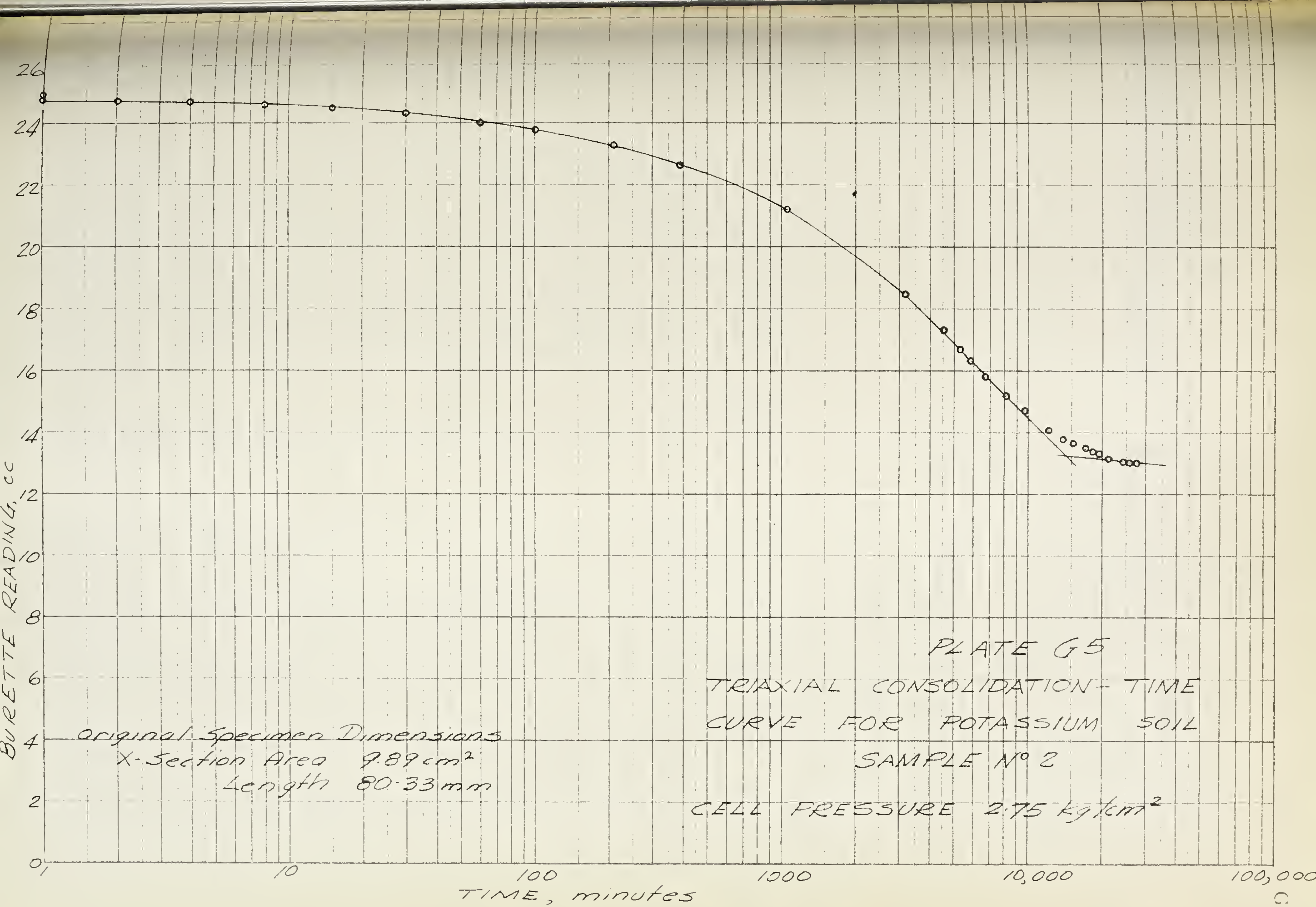


PLATE G5

TRIAXIAL CONSOLIDATION - TIME CURVE FOR POTASSIUM SOIL

SAMPLE Nº 2

CELL PRESSURE 2·75 kg/cm²

Original Specimen Dimensions
X-Section Area 9·89 cm²
Length 80·33 mm

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Potassium (Gp 1)
Depth Sample 2
Engineer ST
Technician DW
Date of Test 8 JUN 61
Test Lateral Pressure σ_3 2.75 kg/cm²
Back Pressure 2.0
Remarks Proving Ring 3-1498
Rate of strain 600 min/10 mm

Area Correction Factor 1.116

a at Failure
$_1 - \sigma_3) =$ 1.90 $\sigma_1 =$ 4.65
1.46 kg/cm²
3.19
1.29
in 9.80 %

me 10 00	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1 - \sigma_3}$
0	0	9.89			0	+0.05					
7	6	9.89	20	.0525	.12	+.05	2.82	2.70	1.04	.07	41.7
9	7	9.89	30	.05165	.17	+.04	2.88	2.71	1.06	.09	23.5
0	8	9.90	40	.05106	.23	+.06	2.92	2.69	1.09	.1	26.1
27	16	9.91	125	.04784	.67	+.19	3.23	2.56	1.26	.2	28.3
37	24	9.92	169	.04682	.89	.28	3.36	2.47	1.36	.3	31.5
45	32	9.93	194	.04639	1.01	.37	3.39	2.38	1.42	.4	33.0
53	40	9.94	213	.04610	1.10	.43	3.42	2.32	1.47	.5	39.1
0	48	9.95	225	.04591	1.16	.47	3.44	2.28	1.51	.6	40.5
7	56	9.96	236	.04577	1.21	.53	3.43	2.22	1.55	.7	43.8
73	64	9.97	244	.04565	1.25	.56	3.44	2.19	1.57	.8	44.8
81	72	9.98	253	.04554	1.29	.60	3.44	2.15	1.60	.9	46.5
7	80	9.99	261	.04546	1.32	.63	3.44	2.12	1.62	1.0	47.7
93	88	10.00	267	.04540	1.35	.66	3.44	2.09	1.65	1.1	48.9
99	96	10.01	273	.04534	1.38	.70	3.43	2.05	1.67	1.2	50.7
05	104	10.02	278	.04528	1.40	.72	3.43	2.03	1.69	1.3	51.4
11	112.5	10.03	283	.04523	1.42	.75	3.42	2.00	1.71	1.4	52.8
16	120.5	10.04	287	.04517	1.44	.77	3.42	1.98	1.73	1.5	53.5
22	128.5	10.05	291	.04514	1.46	.79	3.42	1.96	1.74	1.6	54.1
28	136.5	10.06	295	.04510	1.48	.81	3.42	1.94	1.76	1.7	54.7
35	144.5	10.07	299	.04505	1.49	.85	3.39	1.90	1.78	1.8	57.0
41	153	10.08	303	.04502	1.51	.89	3.37	1.86	1.81	1.9	58.9
48	161	10.09	307	.04499	1.53	.91	3.37	1.84	1.83	2.00	59.4
53	169	10.10	311	.04495	1.54	.91	3.38	1.84	1.84	2.1	59.1
59	177	10.11	313	.04493	1.55	.91	3.39	1.84	1.84	2.2	58.7
64	185	10.12	317	.04489	1.57	.93	3.39	1.82	1.86	2.3	59.2
70	193	10.13	320	.04487	1.58	.94	3.39	1.81	1.87	2.4	59.4
75	201	10.14	323.5	.04483	1.60	.96	3.39	1.79	1.89	2.5	60.0
82	209	10.15	327	.04480	1.61	.98	3.38	1.77	1.91	2.6	60.9

Sheet 2
G14

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Potassium (Gp1)

Depth ______ Sample 2

Area Correction Factor 1.116

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress $\overline{\sigma_1}$ Major	Effective Stress $\overline{\sigma_3}$ Minor	Stress Ratio $\frac{\overline{\sigma_1}}{\overline{\sigma_3}}$	Axial Comp. Strain %	$\overline{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$
225	10.175	332	.04477	1.63	1.03	3.35	1.72	1.95	2.8	63.2
241	10.19	336	.04472	1.65	1.03	3.37	1.72	1.96	3.0	62.4
257	10.21	341	.04468	1.66	1.08	3.33	1.67	1.99	3.2	65.1
273	10.235	346	.04463	1.68	1.10	3.33	1.65	2.02	3.4	65.5
289	10.26	350	.04460	1.70	1.11	3.34	1.64	2.04	3.6	65.3
305.5	10.28	354.5	.04459	1.72	1.14	3.33	1.61	2.07	3.8	66.3
321.5	10.30	358.5	.04453	1.73	1.16	3.32	1.59	2.09	4.0	67.1
337	10.32	361.5	.04450	1.74	1.18	3.31	1.57	2.11	4.2	67.8
353.5	10.345	365.5	.04447	1.75	1.19	3.31	1.56	2.12	4.4	68.0
369.5	10.37	369	.04443	1.76	1.20	3.31	1.55	2.14	4.6	68.2
385.5	10.39	372.5	.04441	1.78	1.22	3.31	1.53	2.16	4.8	68.5
402	10.41	375.5	.04439	1.79	1.23	3.31	1.52	2.18	5.0	68.7
418	10.437	378.5	.04437	1.80	1.25	3.30	1.50	2.20	5.20	69.4
434	10.46	381.5	.04435	1.81	1.24	3.32	1.51	2.20	5.4	68.5
450	10.48	385	.04432	1.82	1.23	3.34	1.52	2.20	5.6	67.5
466	10.50	387	.04430	1.82	1.29	3.28	1.46	2.25	5.8	70.9
482	10.52	389.5	.04429	1.83	1.29	3.29	1.46	2.25	6.0	70.5
498	10.55	392	.04427	1.84	1.29	3.30	1.46	2.26	6.2	70.1
514	10.57	394	.04426	1.84	1.35	3.24	1.40	2.31	6.4	73.4
530	10.59	396	.04424	1.85	1.35	3.25	1.40	2.32	6.6	73.0
546.5	10.61	400	.04421	1.86	1.34	3.27	1.41	2.32	6.8	72.0
562.5	10.64	402	.04420	1.86	1.33	3.28	1.42	2.31	7.00	71.5
578.5	10.66	404	.04419	1.87	1.36	3.26	1.39	2.35	7.2	72.7
771.5	10.94	423	.04405	1.90	1.56★	3.09	1.19	2.6	9.6	82.1
787.5	10.96	423.5	.04405	1.90	1.46	3.19	1.29	2.47	9.8	76.8
803.5	10.99	424.5	.04404	1.90	1.45	3.20	1.30	2.46	10.0	76.3
819.5	11.015	425	.04404	1.90	1.43	3.22	1.32	2.44	10.2	75.2
835.5	11.40	425	.04404	1.83	1.43	3.15	1.32	2.39	10.4	78.1
851.5	11.65	425.5	.04403	1.79	1.43	3.11	1.32	2.36	10.6	

★ Reading unreliable

Content – Initial 53.77

Final 40.85

Saturation – Initial 98.8

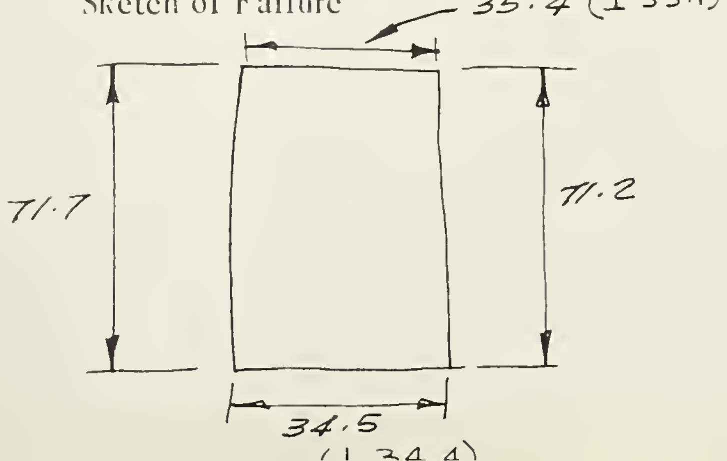
Final 99.1

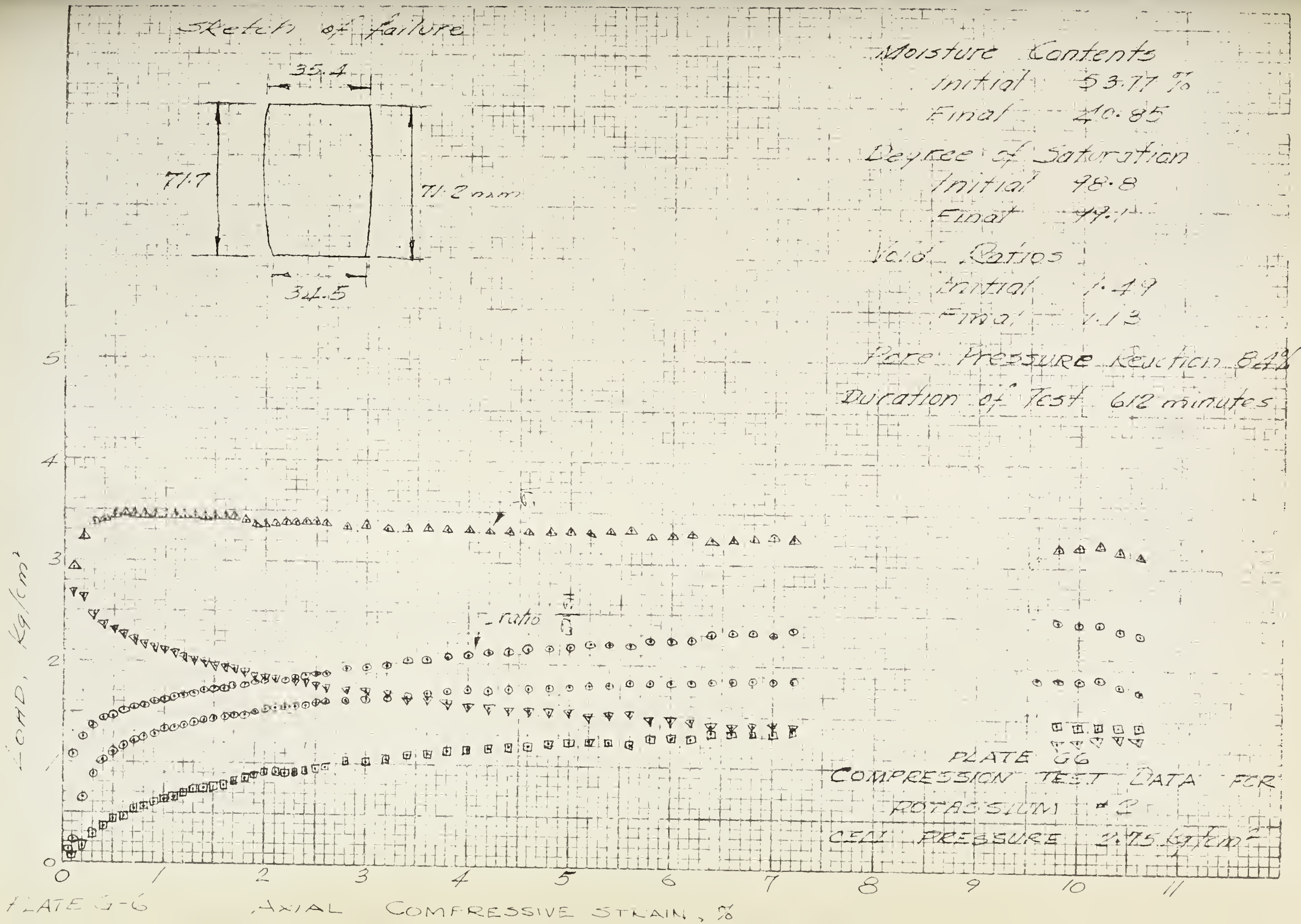
o – Initial 1.49

Final 1.130

ssure Reaction 84%

Sketch of Failure





Moisture Contents
Initial 53.77 %
Final 40.85

Degree of Saturation
Initial 98.8
Final 99.1

Void Ratios
Initial 1.49
Final 1.13

Pore Pressure Reaction 84%
Duration of Test 612 minutes

PLATE G6
COMPRESSION TEST DATA FOR
POTASSIUM #2
CELL PRESSURE 2.75 kg/cm²

PLATE G-6

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Potassium (Gp1)
Depth sample 3
Engineer ST
Technician DW
Date of Set-up 23 May 61
Test Lateral Pressure 4.0 kg/cm²

mple Description CEC = 29.0, S.C. 31.8
e 18.6, Na 0.5, K 75.8, Mg 5.1

5 Wool wicks
itial Weight 134.500 gms

gth, mm	1. 80.36	2. 80.50	Aver: 80.43	Area Top	A_T 9.803	sq.mm
. mm Top	1. 35.14	2. 35.52	Aver. 35.33	Area Centre	A_C 9.959	sq.mm
Centre	1. 35.60	2. 35.62	Aver: 35.61	Area Bottom	A_B 10.049	sq.mm
Bottom	1. 35.82	2. 35.72	Aver: 35.77	Average X-Sect Area		

nal Volume 79.95 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

= ~~7.12~~ 9.94

SOLIDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
ay 1530	0	24.80	
	.1	24.70	
	.25	24.70	
	.5	24.65	
	1	24.60	
	2	24.55	
	4	24.47	
	6	24.40	
	9	24.30	
	15	24.18	
	20	24.10	
	25	24.00	
	30	23.92	
	45	23.72	
	60	23.58	
	100	23.21	
2110	340	21.98	
0845	1035	19.80	
1650	1520	18.80	
845	2475		

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
26 May 0845	3915	15.55	
28 ✓ 0050	6320	13.88	
29 ✓ 0835	8225	13.20	
30 ✓ 0900	9,690	12.80	
31 ✓ 0940	11,175	12.52	
1 JUN 0840	12,555	12.35	
2 ✓ 0840	13,990	12.20	
3 ✓ 1320	15,710	12.10	
5 ✓ 0830	18,300	12.00	
6 ✓ 0830	$19,7\overset{4}{2}0$	11.93	
7 ✓ 0830	20,910	11.92	
8 ✓ 0830	22,350	11.95	
9 JUN 0840	23,800	12.00	

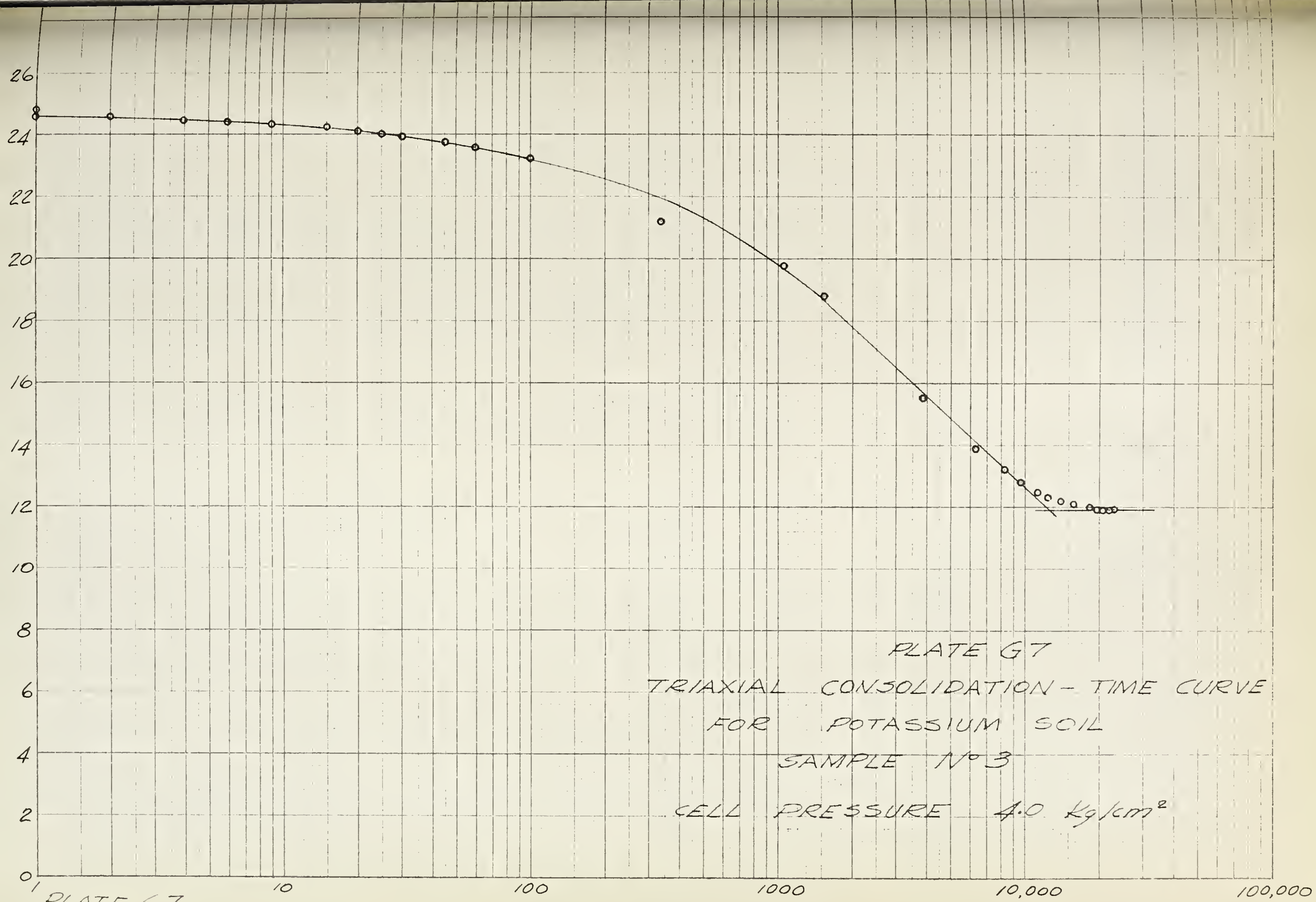


PLATE G7

TRIAXIAL CONSOLIDATION - TIME CURVE
FOR POTASSIUM SOIL
SAMPLE N° 3
CELL PRESSURE 4.0 Kg/cm²

PLATE G7

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Potassium (Gp I)
Depth Sample 3
Engineer ST
Technician DW
Date of Test 12 JUN 61
Test Lateral Pressure σ_3 = 4 kg/cm²
Back Pressure = 2 "
Remarks Proving Ring 1498
Rate of Strain 600 min/10 mm

Area Correction Factor K = 1.126

t Failure
σ_3) = 2.40 σ_1 = 6.40
2.23 kg/cm²
4.17
1.77
10.5 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_P}{\sigma_1 - \sigma_3}$ %
1	9.94	20	.0525	.12	+.23	3.89	3.77	1.03	.01	
1.5	9.94	40	.0511	.23	+.30	3.93	3.70	1.06	.02	
4	9.95	80	.0493	.45	+.36	4.09	3.64	1.12	.05	
8	9.95	123	.0479	.67	.44	4.23	3.56	1.19	.10	65.0
16	9.96	190	.04644	1.00	.57	4.43	3.43	1.29	.20	57.0
24	9.97	233	.04580	1.20	.71	4.49	3.29	1.36	.30	59.2
32	9.98	259	.04548	1.33	.78	4.55	3.22	1.41	.4	58.6
40	9.99	279	.04527	1.42	.86	4.56	3.14	1.45	.5	60.5
48	10.00	294	.04511	1.49	.95	4.54	3.05	1.49	.6	63.7
56	10.01	306	.04500	1.55	.97	4.58	3.03	1.51	.7	62.6
64	10.02	318	.04489	1.60	1.03	4.57	2.97	1.54	.8	64.4
72	10.03	328	.04480	1.65	1.07	4.58	2.93	1.56	.9	64.8
80.5	10.04	338	.0447	1.69	1.09	4.60	2.91	1.58	1.00	64.4
88.5	10.05	347	.04463	1.74	1.17	4.57	2.83	1.61	1.10	67.2
96.5	10.06	353	.04458	1.76	1.24	4.52	2.76	1.64	1.20	70.5
104.5	10.07	360	.04451	1.79	1.26	4.53	2.74	1.65	1.30	70.4
112.5	10.08	366	.04447	1.82	1.32	4.50	2.68	1.68	1.40	72.5
120.5	10.09	373	.04442	1.85	1.33	4.52	2.67	1.69	1.50	71.8
128.5	10.10	379	.04436	1.87	1.37	4.50	2.63	1.71	1.60	73.3
136.5	10.11	384	.04432	1.90	1.38	4.52	2.62	1.73	1.70	72.6
145	10.12	389	.04429	1.92	1.45	4.47	2.55	1.75	1.80	75.5
153	10.13	394	.04426	1.94	1.47	4.47	2.53	1.77	1.90	75.8
161	10.14	398	.04422	1.95	1.50	4.45	2.50	1.78	2.00	76.9
169	10.15	402	.04420	1.97	1.53	4.44	2.47	1.80	2.10	77.7
177	10.16	406	.04416	1.99	1.54	4.45	2.46	1.81	2.2	77.3
165	10.17	411	.04414	2.01	1.54	4.47	2.46	1.82	2.3	76.6
193	10.18	414	.04411	2.02	1.60	4.42	2.40	1.84	2.4	79.2
201	10.19	418	.04408	2.04	1.62	4.42	2.38	1.86	2.5	79.5
209	10.20	421.5	.04406	2.05	1.63	4.42	2.37	1.86	2.6	79.5

Sheet 2
G19

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Potassium (1)

Depth ______ Sample 3

Area Correction Factor 1.126

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\,P_F}{\sigma_1-\sigma_3}$
217	10.21	425	.04404	2.06	1.65	4.41	2.35	1.88	2.70	80.1
225	10.225	428.5	.04400	2.08	1.70	4.38	2.30	1.90	2.80	81.7
233	10.23	432	.0440	2.09	1.72	4.37	2.28	1.92	2.90	82.3
241	10.24	435	.04398	2.10	1.73	4.37	2.27	1.93	3.00	82.4
257.5	10.265	442	.04393	2.13	1.76	4.37	2.24	1.95	3.20	82.6
273.5	10.285	448	.04390	2.15	1.79	4.36	2.21	1.97	3.40	83.3
289.5	10.31	451.	.04389	2.16	1.81	4.35	2.19	1.99	3.60	83.7
305.5	10.33	456	.04386	2.18	1.84	4.34	2.16	2.01	3.80	85.2
322	10.35	461	.04382	2.20	1.86	4.34	2.14	2.03	4.00	84.5
338	10.375	467	.04379	2.22	1.87	4.35	2.13	2.04	4.2	84.2
354	10.395	471	.04376	2.23	1.89	4.34	2.11	2.06	4.4	84.8
370	10.42	475	.04375	2.25	1.91	4.34	2.09	2.08	4.6	84.9
386	10.44	479	.04372	2.26	1.91	4.35	2.09	2.08	4.80	84.5
410	10.475	484	.04369	2.27	1.93	4.34	2.07	2.10	5.1	85.0
418	10.48	486	.04368	2.28	1.93	4.35	2.07	2.1	5.2	84.6
434	10.51	491	.04365	2.30	1.98	4.32	2.02	2.14	5.4	86.1
450	10.53	494	.04363	2.30	1.98	4.32	2.02	2.14	5.6	86.1
466.5	10.55	497	.04361	2.31	2.01	4.30	1.99	2.16	5.8	87.0
483	10.575	501	.04360	2.33	2.05	4.28	1.95	2.19	6.0	88.0
498.5	10.60	503	.04358	2.33	2.05	4.28	1.95	2.19	6.2	88.0
523	10.63	507	.04356	2.34	2.04	4.30	1.97	2.19	6.5	87.2
563	10.69	513	.04353	2.35	2.06	4.29	1.94	2.21	7.0	87.7
603	10.75	521	.04350	2.37	2.14	4.23	1.86	2.27	7.5	90.3
643.5	10.8	527	.04346	2.39	2.11	4.28	1.89	2.26	8.0	88.3
844.5	11.10	545.5	.04338	2.40	2.23	4.17	1.77	2.36	10.5	92.9
885	11.17	547	.04336	2.39	2.27	4.12	1.73	2.37	11.0	95.0
925	11.23	547.5	.04336	2.38	2.25	4.13	1.75	2.36	11.5	94.5
965	11.29	546	.04337	2.36	2.25	4.11	1.75	2.35	12.00	95.3
1005	11.36	540	.04340	2.32	2.21	4.11	1.79	2.30	12.50	95.3

Content – Initial 54.05

Final 39.17%

of Saturation – Initial 98.1

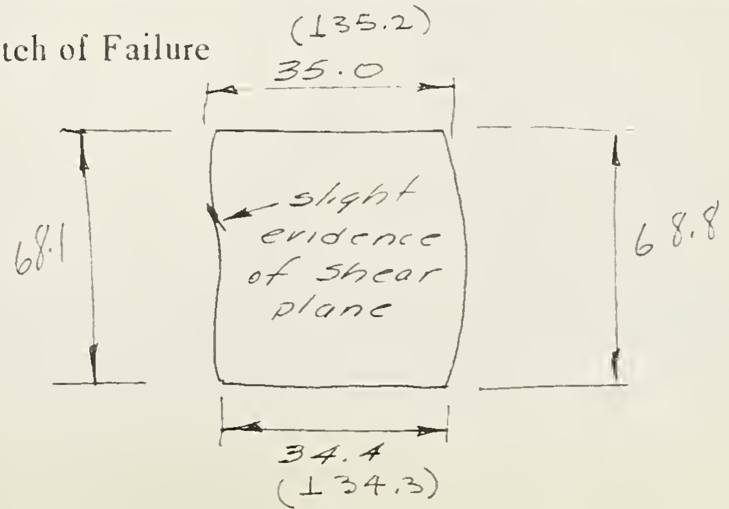
Final 99.3

tio – Initial 1.509

Final 1.081

ssure Reaction 99%

Sketch of Failure



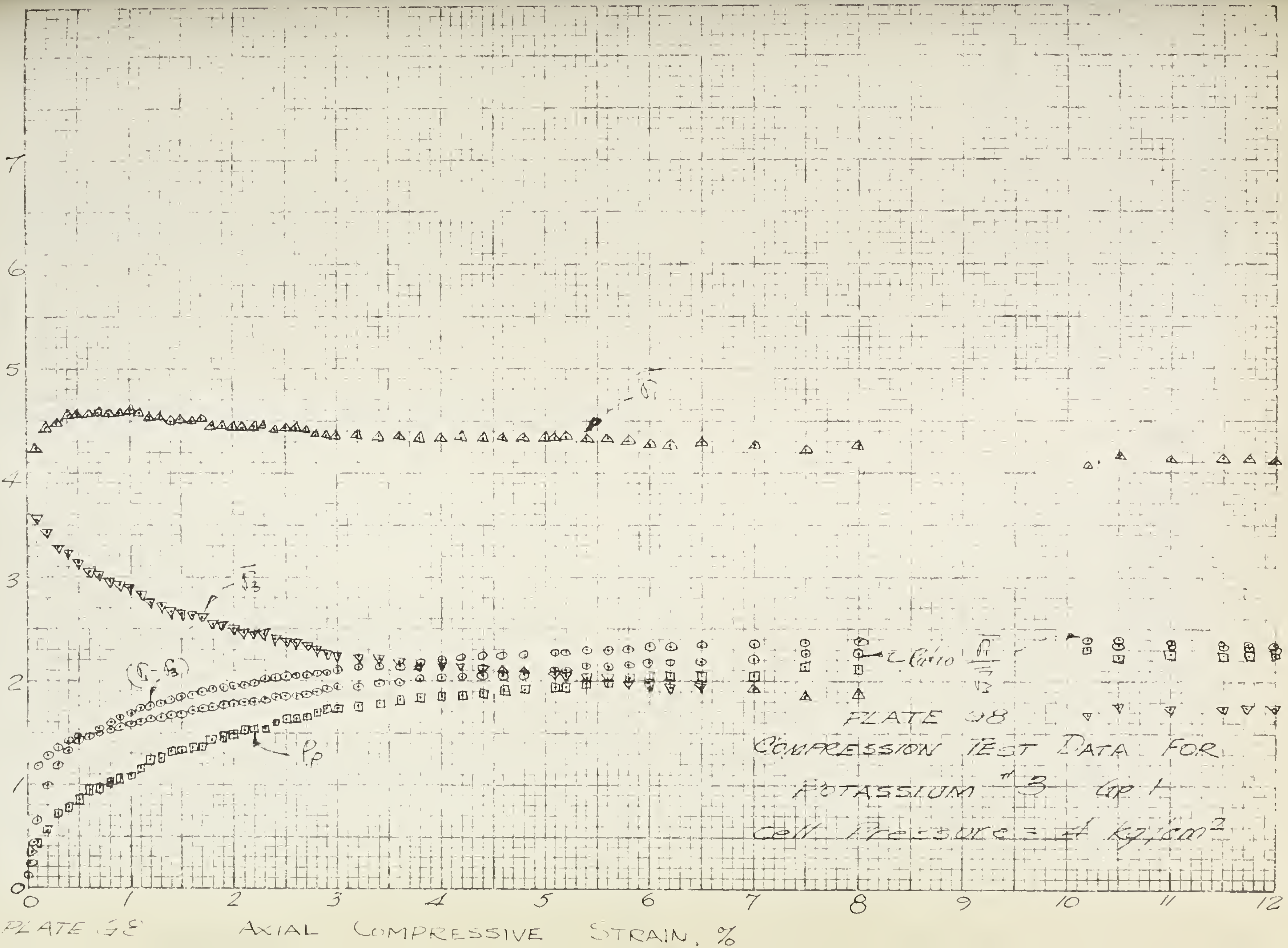


PLATE 38

COMPRESSION TEST DATA FOR

POTASSIUM #3 Gp 1

Cell Pressure = 4 kg/cm²

PLATE 38

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

mple Description Cation Exchange pacity 29.6 me./100 gm Soil; Average ation Analysis: Ca 1.7%, Na 0%, k 95.3%, Mg 3.0; Salt content f pore water 5.1 me/100 gm Soil

itial Weight 136.286 gms

Hole No. Potassium Gp 2

Depth ____ sample 4

Engineer ST

Technician DW

Date of Set-up 8 Aug 61

Test Lateral Pressure 5.25 kg/cm²

gth, mm	1. 80.46	2. 80.44	Aver: 80.45	Area Top	A_T 10.044	sq.mm
m. mm Top	1. 35.80	2. 35.72	Aver: 35.76	Area Centre	A_C 9.937	sq.mm
Centre	1. 35.54	2. 35.60	Aver: 35.57	Area Bottom	A_B 9.831	sq.mm
Bottom	1. 35.40	2. 35.38	Aver: 35.38	Average X-Sect Area		

inal Volume 79.94 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$=$ 9.94 cm²

SOLIDATION DATA

e e	Δt min.	Burette Rdg. c.c.	ΔV c.c.
0930	0	24.50	
	.1	23.90	
	.25	23.80	
	.5	23.75	
	1	23.70	
	2	23.50	
	4	23.45	
	6	23.25	
	8	23.15	
	10	23.05	
	15	22.90	
	20	22.75	
	35	22.35	
	65	21.85	
	100	21.35	
	150	20.75	
	250	19.80	
	300	19.35	
1650	440	18.25	
2100	690	16.72	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
9 Aug 0835	1385	13.90	
1155	1585	13.35	
1640	1870	12.70	
2025	2095	12.30	
2200	2190	12.15	
10 Aug 0835	2825	11.33	
11 " 0845	4275	10.31	
12 " 1145	5895	9.83	
13 " 1530	7560	9.62	
14 " 0830	8580	9.~~62~~ 59	
15 " 0820	10,010	9.50	

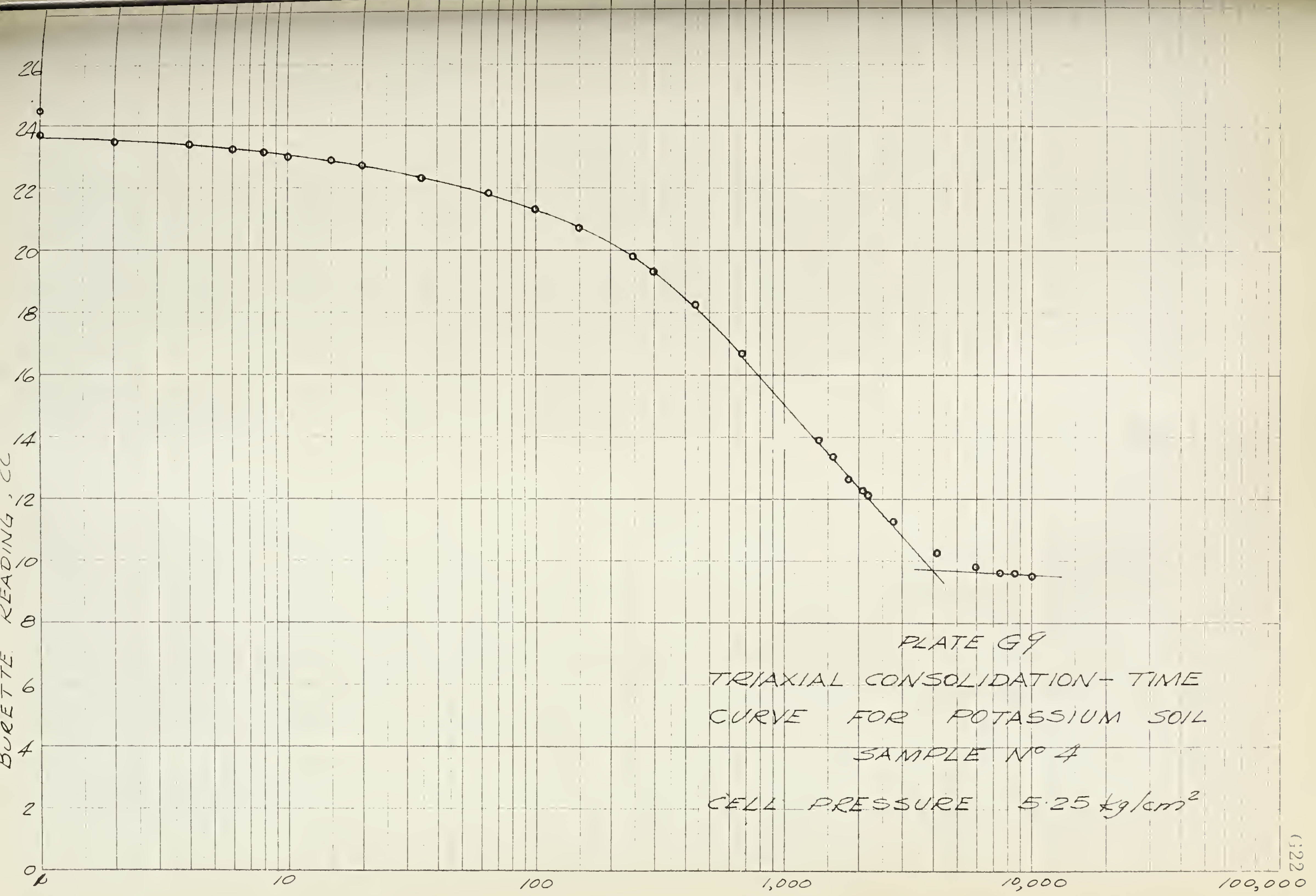


PLATE G9

TRIAXIAL CONSOLIDATION - TIME CURVE FOR POTASSIUM SOIL

SAMPLE N° 4

CELL PRESSURE 5·25 kg/cm^2

Sheet 1

G23

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

L COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Potassium Gp 2

Depth Sample 4

Engineer ST

Technician DW

Date of Test 16 Aug 61

Test Lateral Pressure σ_3 5.25 kg/cm²

Back Pressure 2 kg/cm²

Remarks Proving Ring 3-1498
Rate of Strain 552.3 minutes/10mm

Area Correction Factor 1.155

ailure

σ_3) = 3.45 σ_1 = 8.70

3.13 Kg/cm²

5.57 "

2.12

12.0 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\bar{A} = \frac{P_p}{\sigma_1 - \sigma_3}$ %
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
0.	9.94	0		0						
1	✓	20	.0525	.12	+.04				.01	
2.5	✓	40	.0511	.24	+.04				.03	
4	✓	60	.0501	.35	.04				.05	
8	9.95	125	.04785	.69	.09	5.85	5.16	1.13	.10	13.0
16	9.96	225	.04591	1.20	.29	6.16	4.96	1.24	.20	24.2
24	9.97	285	.0452	1.49	.46	6.28	4.79	1.31	.30	30.9
32	9.98	325	.04482	1.69	.64	6.30	4.61	1.37	.4	37.8
40	9.99	357	.04453	1.84	.78	6.31	4.47	1.41	.5	42.4
48.5	10.00	383.5	.04434	1.96	.92	6.29	4.33	1.45	.6	46.9
56.5	10.01	405	.04418	2.06	1.03	6.28	4.22	1.49	.7	50.0
64.5	10.02	423	.04404	2.15	1.14	6.26	4.11	1.52	.8	53.0
72.5	10.03	439.5	.04395	2.22	1.20	6.27	4.05	1.55	.9	54.1
80.5	10.04	455	.04386	2.30	1.32	6.23	3.93	1.59	1.0	57.3
88.5	10.05	468.5	.04378	2.36	1.43	6.18	3.82	1.62	1.1	60.5
96.5	10.06	481	.04370	2.41	1.48	6.18	3.77	1.64	1.2	61.4
104.5	10.07	491	.04364	2.46	1.55	6.16	3.70	1.66	1.3	63.0
112.5	10.08	502.5	.04359	2.51	1.64	6.12	3.61	1.70	1.4	65.3
120.5	10.09	513	.04353	2.56	1.69	6.12	3.56	1.72	1.5	66.0
128.5	10.10	522.5	.04348	2.60	1.78	6.07	3.47	1.75	1.6	68.4
137	10.11	533	.04343	2.64	1.79	6.10	3.46	1.76	1.7	67.8
145	10.12	542	.04339	2.68	1.86	6.07	3.39	1.79	1.8	69.4
153	10.13	549.5	.04335	2.72	1.90	6.07	3.35	1.81	1.9	69.9
161	10.14	558	.04331	2.75	1.98	6.02	3.27	1.84	2.0	72.0
177	10.16	572.5	.04325	2.81	2.06	6.00	3.19	1.88	2.2	73.3
193	10.185	585	.04320	2.87	2.15	5.97	3.10	1.93	2.4	75.0
209	10.205	597.5	.04314	2.92	2.22	5.95	3.03	1.96	2.6	76.0
225	10.225	608.5	.04310	2.96	2.28	5.93	2.97	2.00	2.8	77.0
241	10.25	618	.04307	3.00	2.35	5.90	2.90	2.03	3.0	78.4

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Potassium Gp 2

Depth Sample 4

Area Correction Factor 1.155

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}}{P_F}$ $\sigma_1 - \sigma_3$
281	10.30	643	.04298	3.10	2.48	5.87	2.77	2.12	3.5	80.0
321	10.355	663.5	.04290	3.17	2.58	5.84	2.67	2.19	4.0	81.4
362	10.41	683	.04283	3.25	2.67	5.83	2.58	2.26	4.5	82.2
402	10.46	701	.04276	3.31	2.76	5.80	2.49	2.33	5.0	83.4
462.5	10.545	722	.04268	3.38	2.87	5.76	2.38	2.42	5.75	84.9
523	10.63	739	.04262	3.42	2.94	5.73	2.31	2.48	6.5	86.0
563	10.69	746.5	.04260	3.43^5	2.97	5.71	2.28	2.504	7.0	86.5
603.5	10.745	752	.04258	3.44	3.00	5.69	2.25	2.529	7.5	87.2
643.5	10.805	758	.04256	3.45	3.06	5.64	2.19	2.575	8.0	88.6
684	10.86	761	.04254	3.44	3.09	5.60	2.16	2.593	9.0	89.8
764	10.98	766	.04252	3.43	3.11	5.57	2.14	2.603	9.5	90.7
893	11.18	781	.04248	3.43	3.12	5.56	2.13	2.610	11.10	92.4
925	11.23	790	.04244	3.45	3.14	5.58	2.11	2.620	11.5	91.0
965.5	11.295	796	.04242	3.45	3.13	5.57	2.12	2.627	12.0	90.7
1007.5	11.36	800	.04240	3.45	3.12	5.58	2.13	2.620	12.5	90.4
1046	11.425	803	.04239	3.44	3.12	5.57	2.13	2.615	13.0	90.6

Content – Initial 51.09

Final 35.07

f Saturation – Initial 97.9%

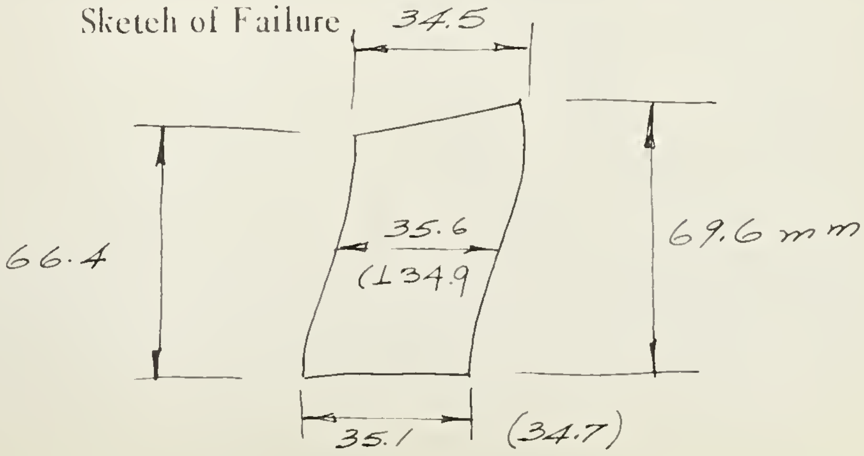
Final 96.1

io – Initial 1.428

Final 0.998

ssure Reaction

Sketch of Failure



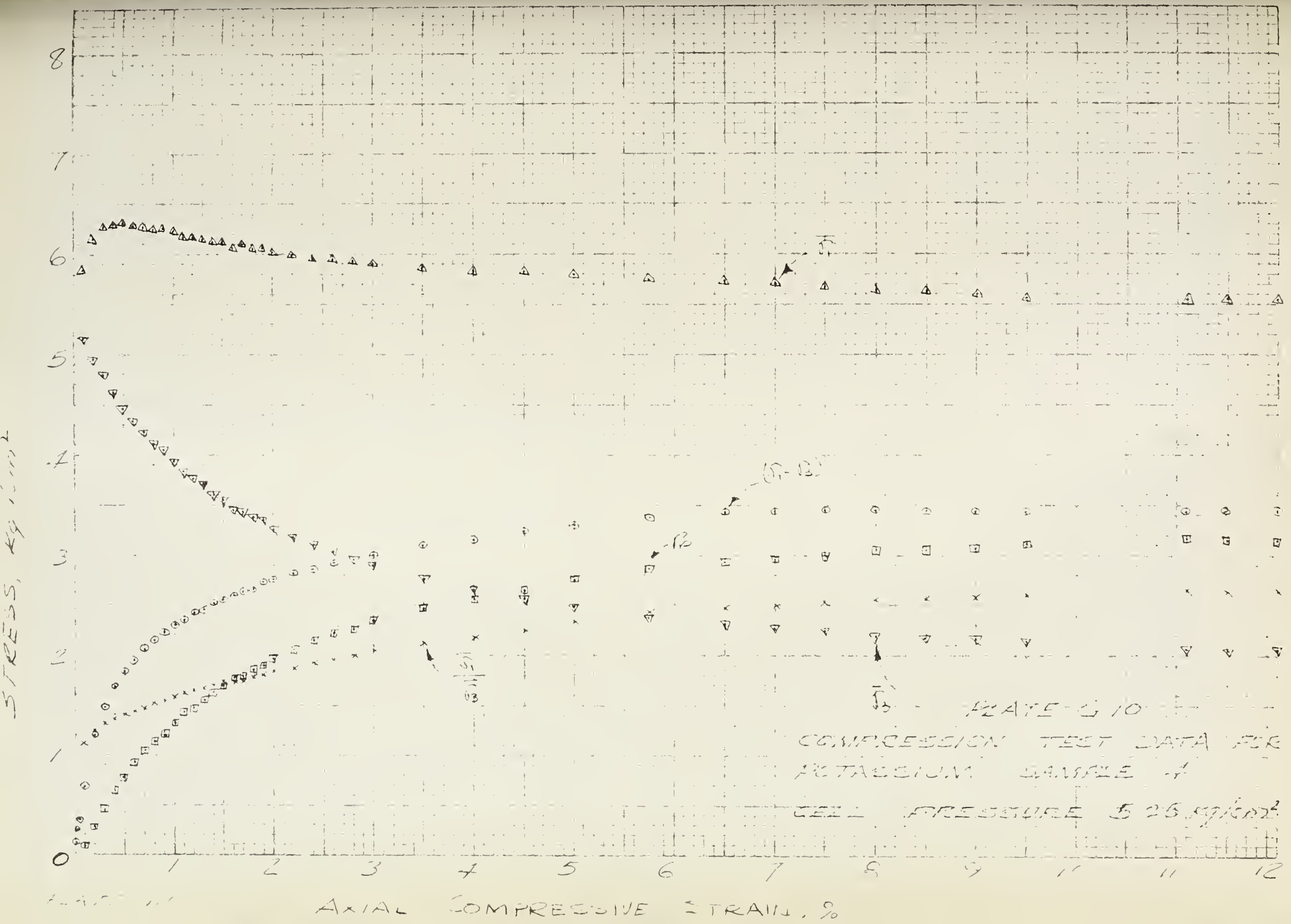


PLATE C 10

COMPRESSION TEST DATA FOR POTASSIUM SAMPLE 4

CELL PRESSURE 5.25 kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

AXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Potassium Gp 2
Depth sample 5
Engineer ST
Technician DW
Date of Set-up 17 Oct 61
Test Lateral Pressure 6·5 kg/cm²

ple Description CEC = 29.6 me/100gm.
rage Exchange Cations
a 1·7%, Na 0%, K=95·3%
g 3·0%, Salt Content 5·1 me/100gm
/.

al Weight ~~136·286~~ 139.642 gms

, mm 1. 80·46 2. 80·44 Aver: 80·45 Area Top A_T 10.089 sq.mm
mm Top 1. 35·82 2. 35·86 Aver: 35·84 Area Centre A_C 9·959 sq.mm
Centre 1. 35·58 2. 35·64 Aver: 35·61 Area Bottom A_B 9.865 sq.mm
Bottom 1. 35·46 2. 35·42 Aver: 35·44 Average X-Sect Area

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 9.97$

l Volume 80·19 cc

LIDATION DATA

	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
020	0	24.63	
	.1	24.40	
	·25	24.37	
	·50	24.30	
	1	24.25	
	2	24.15	
	4	24.00	
	6	23·90	
	8	23·84	
	10	23.75	
	15	23.60	
	20	23.45	
	30	23.25	
	75	22.50	
	90	22.26	
350	210	21.00	
10	350	19.85	
00	400	19.45	
35	555	18·40	
05	645	17·90	

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
18 Oct 0810	1310	15·15	
✓ 1720	1860	13·80	
19 Oct 0930	2830	12·38	
20 ✓ 1615	4675	11·14	
21 ✓ 0830	5650	10·84	
23 ✓ 0830	8530	10·42	
24 ✓ 1130	10,150	10·30	
27 ✓ 1440	14,660	10.15	14·48
		K = 1·146	

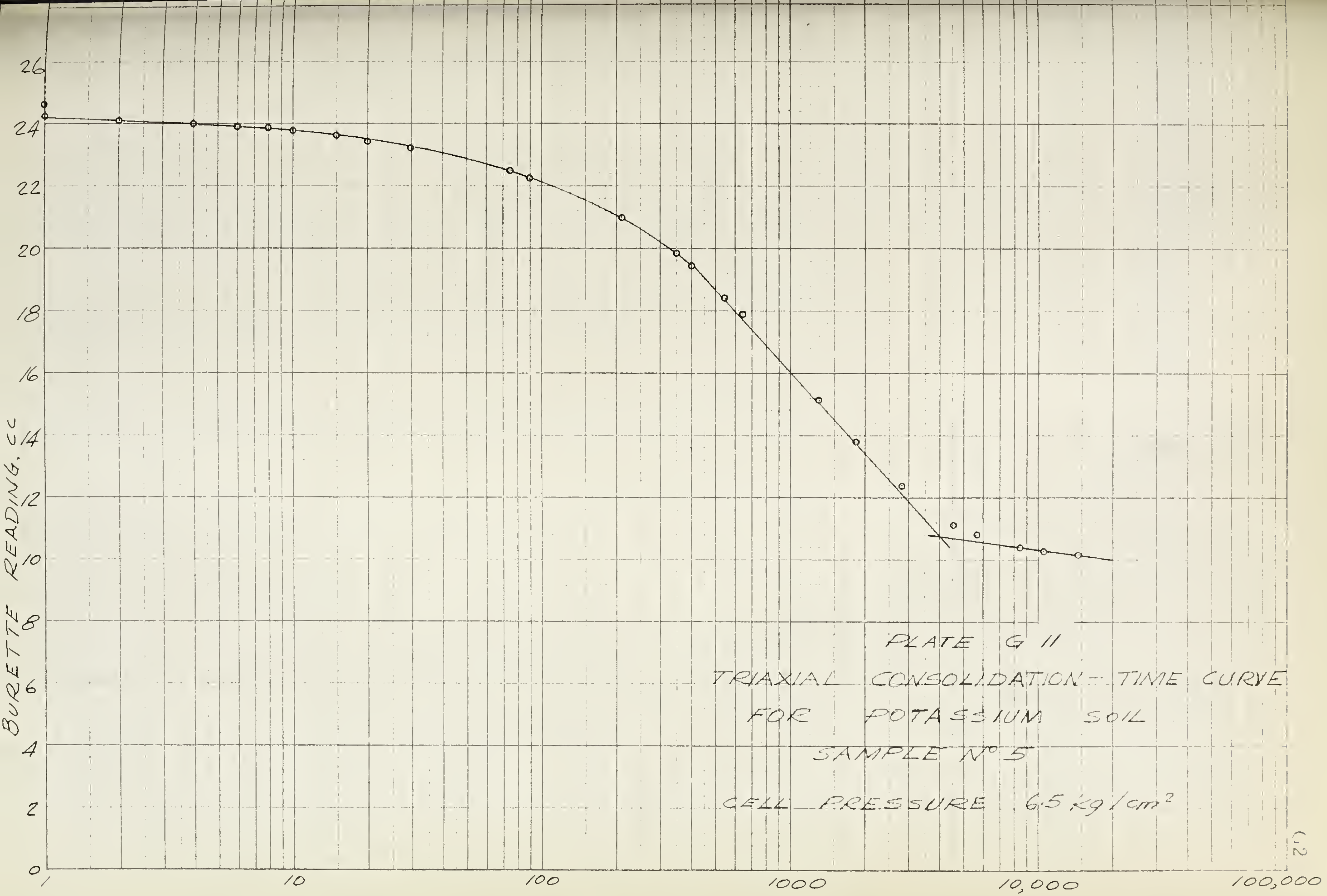


PLATE G 11

TRIAXIAL CONSOLIDATION - TIME CURVE

FOR POTASSIUM SOIL

SAMPLE N° 5

CELL PRESSURE 6.5 kg/cm²

Sheet 1 G28

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. POTASSIUM Gp 2

Depth ______ Sample 5

Engineer ST

Technician —

Date of Test 28 Oct 61

Test Lateral Pressure σ_3 = 6.50 kg/cm²

Back Pressure 2 kg/cm²

Remarks Rate of Strain 362 min/10mm
Proving Ring 3-1498

Area Correction Factor K = 1.146

t Failure

σ_3) = 4.53 σ_1 = 11.03

3.77 kg/cm²

7.26

2.73

12.0 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_p	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
0	9.97			0						
1	9.97	20 ~~057~~	.0525	.12	+.08				.25	
2	✓	40	.0511	.235	.09				.37	
3	✓	60	.0501	.35	.11				.50	
8	✓	163	.04693	.88	.24	7.14	6.26	1.141	.1	27.3
16	9.99	279	.04526	1.45	.51	7.44	5.99	1.242	.2	
24	10.00	353	.04458	1.80	.71	7.59	5.79	1.311	.3	
32	10.01	406	.04418	2.05	.90	7.65	5.60	1.366	.4	43.9
40	10.02	445	.04391	2.23	1.08	7.65	5.42	1.411	.5	48.4
48	10.03	480	.04370	2.40	1.22	7.68	5.28	1.455	.6	50.8
56	10.04	508	.04356	2.53	1.36	7.67	5.14	1.492	.7	53.8
64	10.05	535	.04341	2.65	1.50	7.65	5.00	1.530	.8	56.6
72.5	10.06	560	.04330	2.76	1.61	7.65	4.89	1.564	.9	58.3
80.5	10.07	581	.04321	2.86	1.76	7.60	4.74	1.603	1.0	61.5
88.5	10.08	599	.04314	2.94	1.85	7.59	4.65	1.632	1.1	62.9
96.5	10.09	615	.04309	3.01	1.90	7.61	4.60	1.654	1.2	63.1
105	10.10	630	.04301	3.07	1.99	7.58	4.51	1.681	1.3	64.8
112.5	10.11	644	.04296	3.14	2.07	7.57	4.43	1.709	1.4	65.9
121	10.12	659	.04290	3.20	2.13	7.57	4.37	1.732	1.5	66.6
129	10.13	670	.04288	3.25	2.20	7.55	4.30	1.756	1.6	67.7
137	10.14	684	.04281	3.31	2.30	7.51	4.20	1.788	1.7	69.5
145	10.15	695	.04279	3.36	2.33	7.53	4.17	1.806	1.8	69.3
153	10.16	706	.04274	3.40	2.39	7.51	4.11	1.827	1.9	70.3
161	10.17	715	.04270	3.44	2.43	7.51	4.07	1.845	2.0	70.6
177	10.19	733	.04263	3.51	2.53	7.48	3.97	1.884	2.2	72.1
193	10.22	752	.04259	3.59	2.62	7.47	3.88	1.925	2.4	73.0
209	10.24	766	.04251	3.64	2.72	7.42	3.78	1.963	2.6	74.7
225	10.26	781	.04249	3.71	2.79	7.42	3.71	2.000	2.8	75.2
241	10.28	794	.04241	3.75	2.86	7.39	3.64	2.030	3.0	76.3

Sheet 2
G29

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. POTASSIUM Gp 2

Depth Sample 5

Area Correction Factor 1.146

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg cm² P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ P_F $\sigma_1-\sigma_3$
282	10.33	826	.04230	3.88	3.00	7.38	3.50	2.109	3.5	77.3
322	10.39	854	.04220	3.98	3.12	7.36	3.38	2.178	4.0	78.4
362	10.44	879	.04212	4.06	3.21	7.35	3.29	2.234	4.5	79.1
402	10.495	901	.04205	4.14	3.30	7.34	3.20	2.294	5.0	79.7
442.5	10.55	921	.04199	4.20	3.38	7.32	3.12	2.346	5.5	80.5
483	10.605	939	.04191	4.25	3.45	7.30	3.05	2.393	6.0	81.2
523	10.66	954	.04188	4.30	3.52	7.28	2.98	2.443	6.5	81.9
563	10.72	970	.04181	4.34	3.56	7.28	2.94	2.476	7.0	82.0
603.5	10.78	985	.04177	4.37	3.58	7.29	2.92	2.497	7.5	81.9
643.5	10.835	1000	.04172	4.41	3.66	7.25	2.84	2.553	8	83.0
684	10.895	1012	.04169	4.44	3.68	7.26	2.82	2.574	8.5	82.9
724	10.955	1024	.04165	4.46	3.70	7.26	2.80	2.593	9.0	83.0
764	11.015	1035	.04161	4.48	3.72	7.26	2.78	2.612	9.5	83.0
805	11.08	1046	.04159	4.50	3.71	7.29	2.79	2.613	10.0	82.9
845	11.14	1055	.04156	4.51	3.72	7.29	2.78	2.622	10.5	82.5
885	11.20	1063	.04154	4.52	3.75	7.25	2.75	2.636	11.0	83.0
935	11.28	1071	.04151	4.52	3.76	7.26	2.74	2.650	11.62	83.2
965	11.30	1076	.04150	4.53	3.77	7.26	2.73	2.659	12.0	83.2
1005.5	11.395	1080	.04149	4.51	3.75	7.26	2.75	2.640	12.5	83.1
1046	11.46	1082	.04148	4.49	3.76	7.23	2.74	2.639	13.0	83.7
1078	11.51	1083	4148	4.47	3.77	7.20	2.73	2.637	13.4	84.3

Content – Initial 48.0 %

Final 33.52

Saturation – Initial 99

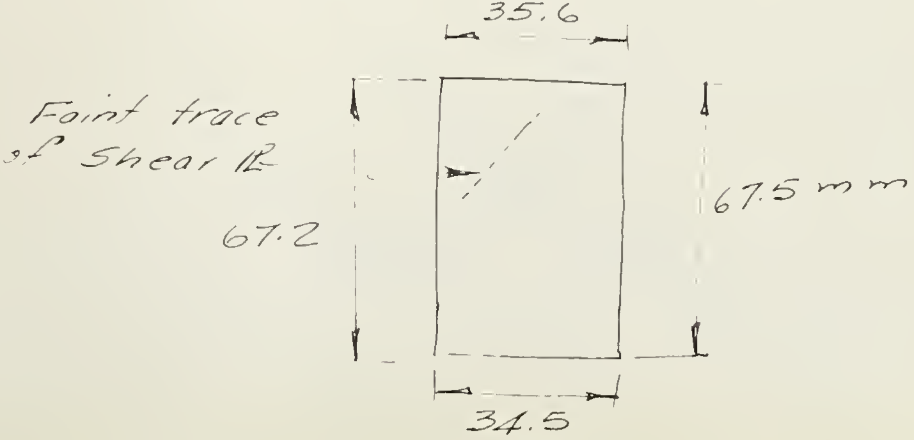
Final 98.6

o – Initial 1.334

Final 0.931

sure Reaction 54%

Sketch of Failure



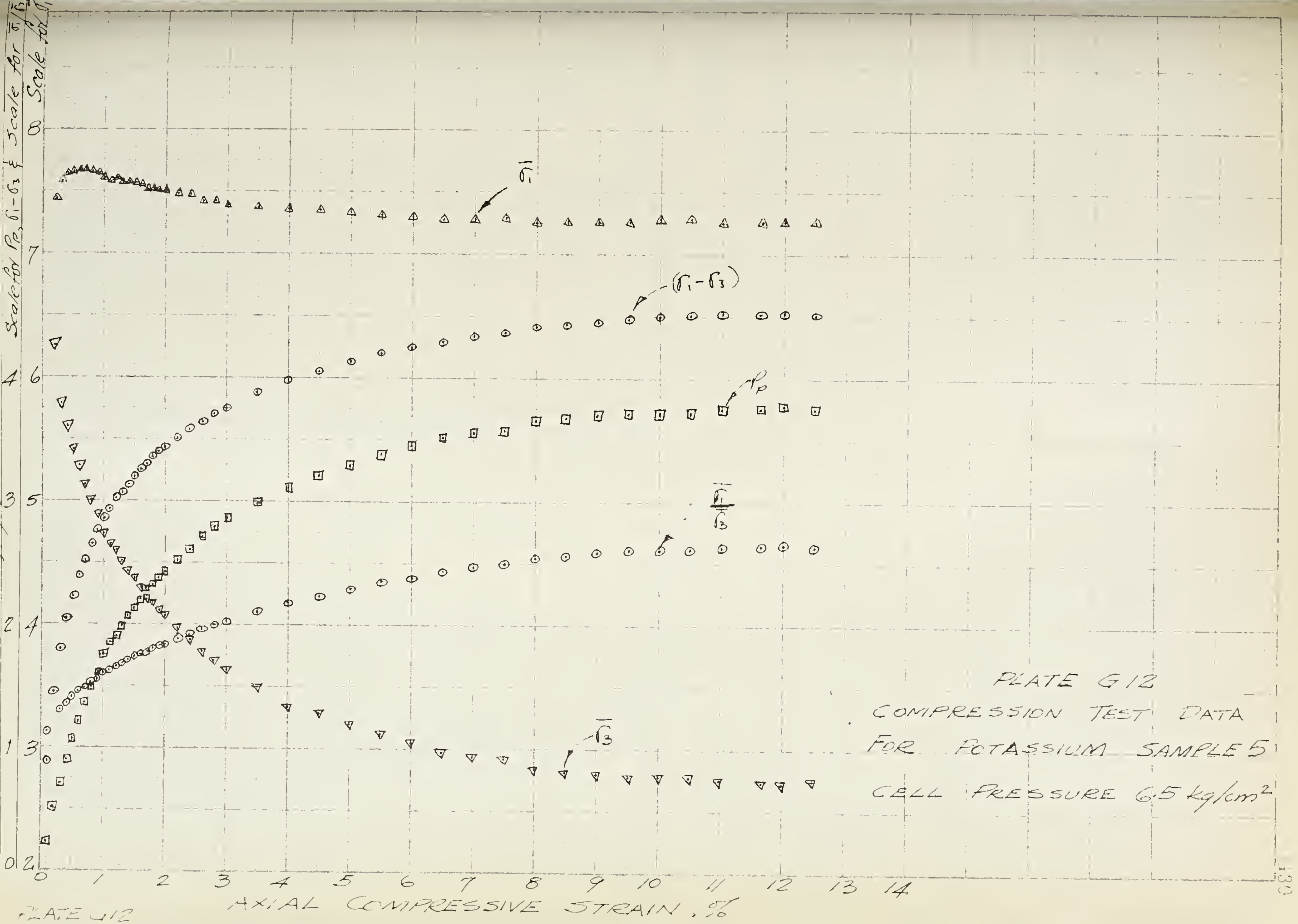


PLATE G12

COMPRESSION TEST DATA

FOR POTASSIUM SAMPLE 5

CELL PRESSURE 6.5 kg/cm²

PLATE G12

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

mple Description CEC = 29.6 m.e./100gm
verage Exchange Cations
a 1.7%, Na 0%, K = 95.3%
g 3.0, Salt Content 5.1 m.e.
er 100 gm soil.

tial Weight 137.778 gms

Project THESIS
Hole No. POTASSIUM Gp 2
Depth ______ sample 6
Engineer ST
Technician DW
Date of Set-up 8 Aug 61
Test Lateral Pressure 7.5 kg/cm²

th, mm 1. 80.52 2. 80.48 Aver: 80.50 Area Top A_T 10.167 sq.mm
. mm Top 1. 36.00 2. 35.96 Aver: 35.98 Area Centre A_C 10.372 sq.mm
Centre 1. 36.50 2. 36.18 Aver: 36.34 Area Bottom A_B 10.463 sq.mm
Bottom 1. 36.52 2. 36.48 Aver: 36.50 Average X-Sect Area

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 10.34$

al Volume 83.24 cc

OLIDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
1100	0	25.00	
	·1	24.60	
	·25	24.60	
	·5	24.55	
	1	24.45	
	2	24.35	
	4	24.20	
	6	24.00	
	8	23.90	
	10	23.80	
	15	23.60	
	20	23.40	
	30	23.10	
	50	22.60	
	150	20.80	
	200	20.20	
50	350	18.50	
00	600	16.50	
840	1300	13.15	
55	1495	12.60	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
9 AUG 1635	1775	12·07	
✓ 2200	2100	11·38	
10 ✓ 0830	2730	10·53	
11 ✓ 0845	4185	9·40	
12 ✓ 1145	5805	8·75	
13 ✓ 1530	7470	8·43	
14 ✓ 0830	8490	8·32	
15 ✓ 0825	9925	8·20	
16 ✓ 0915	11,415	8·11	
16 ✓ 2145	12,165	8.15	16.85
			K = 1.175

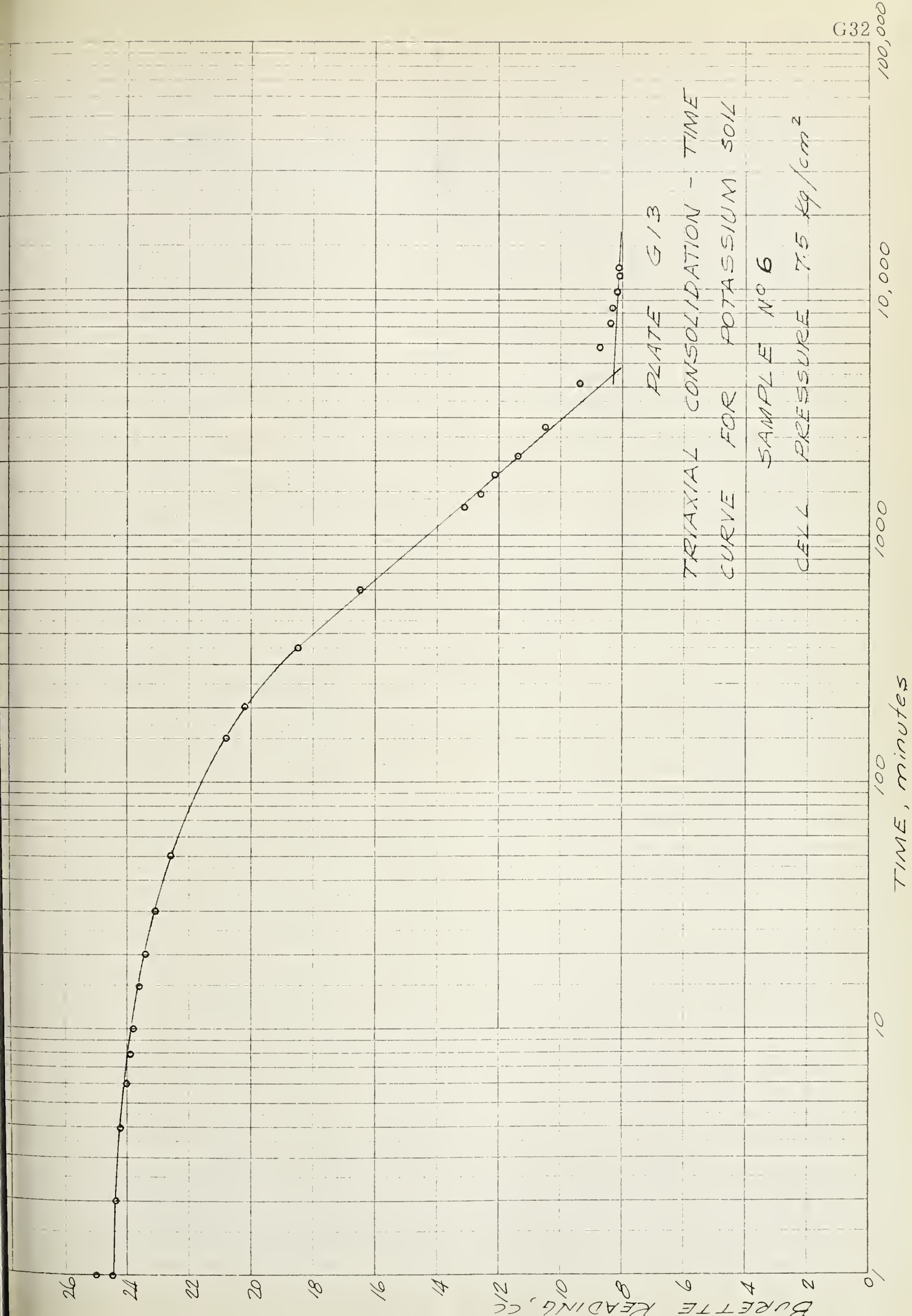


PLATE G13

TRIAXIAL CONSOLIDATION - TIME CURVE FOR POTASSIUM SOIL

SAMPLE No 6

CELL PRESSURE 7.5 kg/cm²

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Potassium Gp2

Depth ______ Sample 6

Engineer ST

Technician DW

Date of Test 17 Aug '61

Test Lateral Pressure σ_3 7.5 kg/cm²

Back Pressure 2.0 kg/cm²

Remarks Proving Ring 3-1498

Rate of Strain

408 min/10mm

Failure

σ_3) = 4.72 σ_1 = 12.22

4.32 kg/cm²

7.90

3.18

12 %

Area Correction Factor 1.175

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$
	10.34			0	+.05					
4	10.34	20	.0525	.12	+.22				.05	
11	10.35	40	.0511	.23	.25				.14	
16	10.36	247	.04563	1.27	.76	8.01	6.74	1.19	.2	
20	10.365	322	.04485	1.63	.97	8.16	6.53	1.25	.25	
28	10.37	409	.04416	2.05	1.20	8.35	6.30	1.32	.35	
32	10.38	441	.04394	2.19	1.31	8.38	6.19	1.35	.4	59.8
40	10.39	491	.04365	2.42	1.46	8.46	6.04	1.40	.5	60.3
48.5	10.40	531	.04344	2.61	1.64	8.47	5.86	1.45	.6	62.8
56.5	10.41	561	.04330	2.74	1.70	8.54	5.80	1.47	.7	62.0
64.5	10.42	587	.04320	2.86	1.78	8.58	5.72	1.50	.8	62.2
72.5	10.43	609	.04310	2.96	1.95	8.51	5.55	1.53	.9	65.9
80.5	10.445	630	.04302	3.05	2.07	8.48	5.43	1.56	1.0	67.9
88.5	10.45	648	.04295	3.13	2.22	8.41	5.28	1.59	1.1	70.9
96.5	10.46	664	.04290	3.20	2.28	8.42	5.22	1.61	1.2	71.3
104.5	10.47	679	.04284	3.26	2.37	8.39	5.13	1.64	1.3	72.7
112.5	10.49	694	.04279	3.33	2.44	8.39	5.06	1.66	1.4	73.3
121	10.50	707	.04275	3.38	2.48	8.40	5.02	1.67	1.5	73.4
129	10.51	720	.04269	3.44	2.57	8.37	4.93	1.70	1.6	74.7
137	10.52	732	.04264	3.49	2.65	8.34	4.85	1.72	1.7	75.9
145	10.53	744	.04260	3.54	2.76	8.28	4.74	1.75	1.8	78.0
153	10.54	755	.04257	3.58	2.78	8.30	4.72	1.76	1.9	77.7
161	10.55	764	.04254	3.62	2.80	8.32	4.70	1.77	2.0	77.3
177	10.57	784	.04247	3.70	2.92	8.28	4.58	1.81	2.2	78.9
193	10.59	802	.04240	3.77	3.01	8.26	4.49	1.84	2.4	79.8
209.5	10.61	818	.04233	3.83	3.08	8.25	4.42	1.87	2.6	80.4
225.5	10.64	833	.04229	3.89	3.20	8.19	4.30	1.90	2.8	82.3
241.5	10.66	847	.04224	3.94	3.34	8.10	4.16	1.95	3.0	84.8
282	10.71	880	.04212	4.06	3.48	8.08	4.02	2.01	3.5	85.7

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Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Potassium Gp2

Depth Sample 6

Area Correction Factor 1.175

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm2 P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\,P_P}{\sigma_1 - \sigma_3}$
322	10.77	907	.04203	4.16	3.58	8.08	3.92	2.06	4.0	86.1
362	10.83	932	.04194	4.24	3.70	8.04	3.80	2.12	4.5	87.3
402.5	10.88	955	.04187	4.32	3.78	8.04	3.72	2.16	5.0	87.5
443	10.94	976	.04180	4.38	3.89	7.99	3.61	2.21	5.5	88.8
483	11.00	994	.04713	4.43	4.00	7.93	3.50	2.27	6.0	90.3
523	11.06	1011	.04710	4.48	4.04	7.94	3.46	2.295	6.5	90.2
56~~0~~3.5	11.12	1026	.04165	4.51	4.07	7.94	3.43	2.315	7.0	90.2
604	11.18	1042	.04160	4.56	4.12	7.94	3.38	2.349	7.5	90.4
644	11.24	1057	.04155	4.59	4.19	7.90	3.31	2.387	8.0	91.3
684	11.30	1070	.04151	4.62	4.22	7.90	3.28	2.409	8.5	91.3
724.5	11.36	1083	.04148	4.65	4.24	7.91	3.26	2.426	9.0	91.2
765	11.425	1097	.04143	4.67	4.27	7.90	3.23	2.446	9.5	91.4
805	11.49	1108	.04140	4.69	4.26	7.93	3.24	2.448	10	90.8
845	11.55	1118	.04138	4.71	4.30	7.91	3.20	2.472	10.5	91.3
885.5	11.62	1128	.04135	4.72	4.30	7.92	3.20	2.475	11.0	91.1
926	11.68	1137	.04132	4.73	4.31	7.92	3.19	2.483	11.5	91.1
966	11.75	1143	.04130	4.72	4.32	7.90	3.18	2.484	12	91.5
1006	11.82	1149	.04130	4.72	4.32	7.90	3.18	2.484	12.5	91.5
1046.5	11.88	1150	.04130	4.70	4.31	7.89	3.19	2.473	13	91.7
1087	11.95	1150	.04130	4.67	4.28	7.89	3.22	2.450	13.5	91.6
1127	12.02	1150	✓	4.64	4.30	7.84	3.20	2.450	14	92.6
1167	12.09	1147	✓	4.60	4.30	7.80	3.20	2.437	14.5	-
1207.5	12.16	1142	✓	4.56	4.26	7.80	3.24	2.407	15	-

ontent – Initial 50.64

Final 32.75

Saturation – Initial 92.9

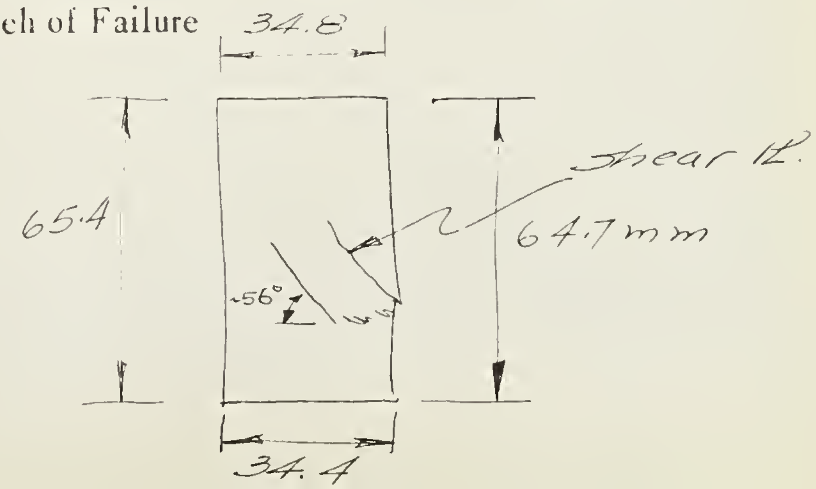
Final 99.1

– Initial 1.494

Final 0.906

ure Reaction 93%

Sketch of Failure



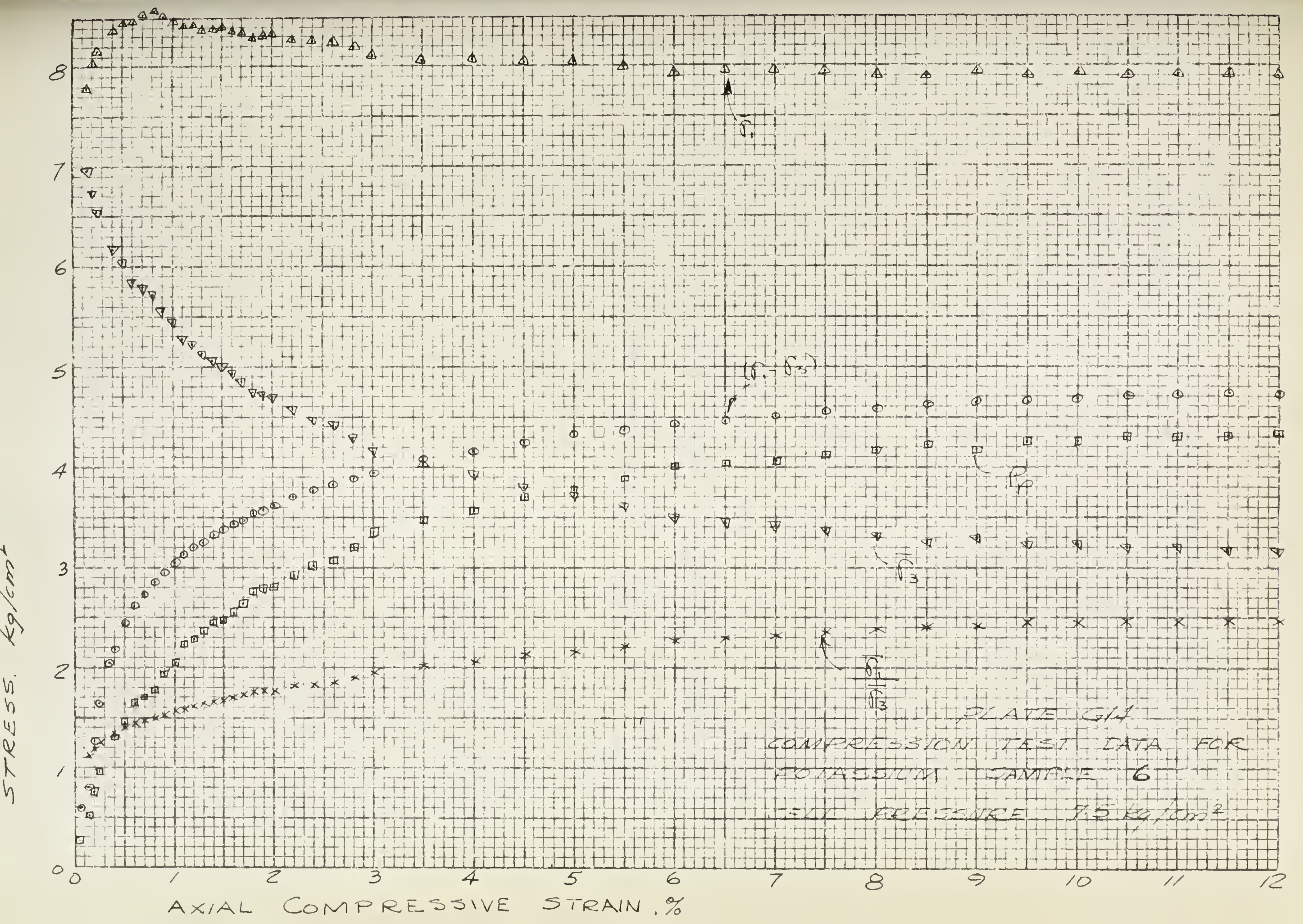


PLATE G14
COMPRESSION TEST DATA FOR
POTASSIUM SAMPLE 6
CELL PRESSURE 7.5 kg/cm²

APPENDIX H

DATA SHEETS FOR TESTS ON CALCIUM MODIFIED SOIL

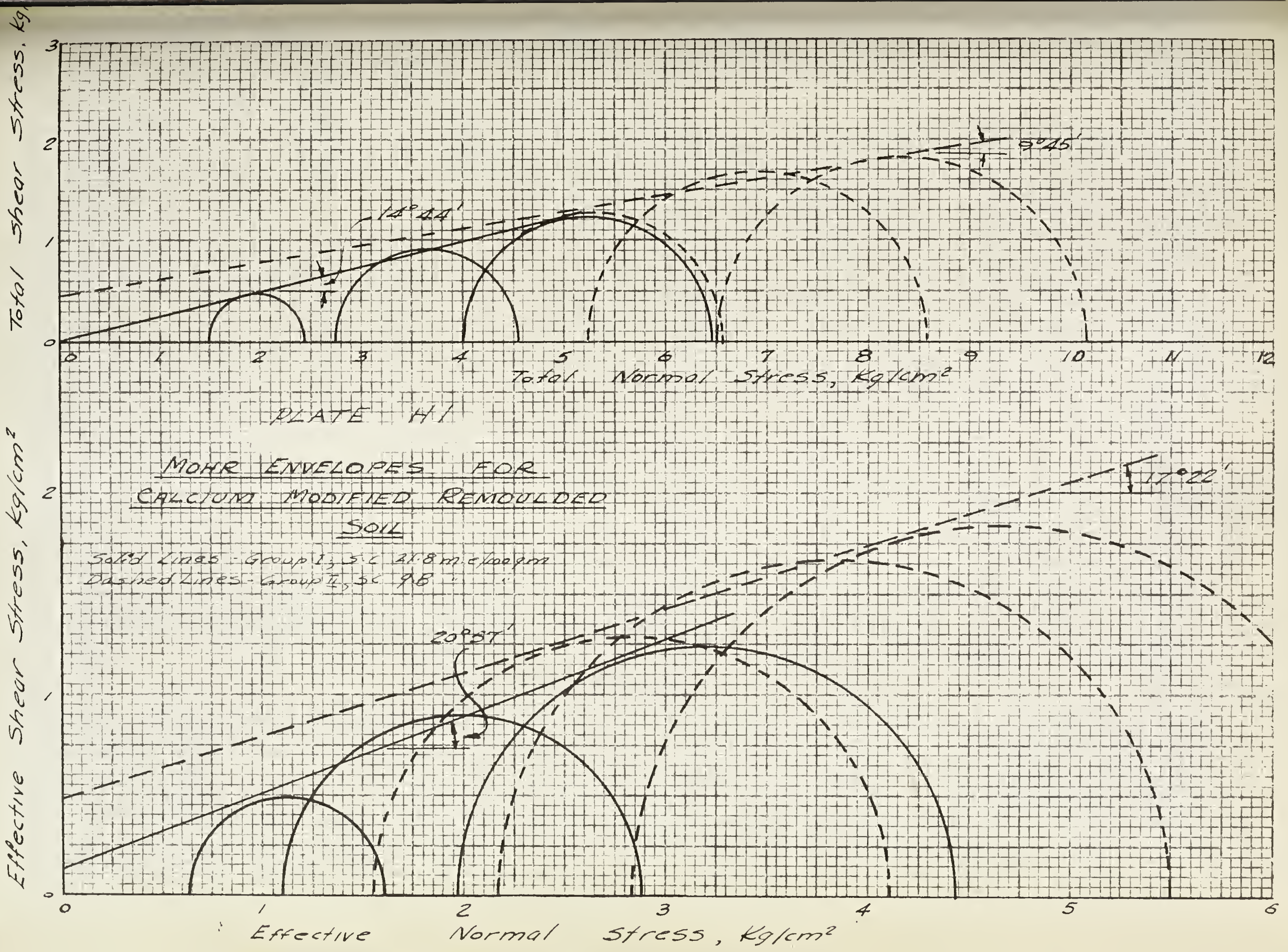


PLATE H1

MOHR ENVELOPES FOR
CALCIUM MODIFIED REMOULDED
SOIL

Solid Lines - Group I, S.C 21·8 m.e/100gm
Dashed Lines - Group II, S.C 9.8 " "

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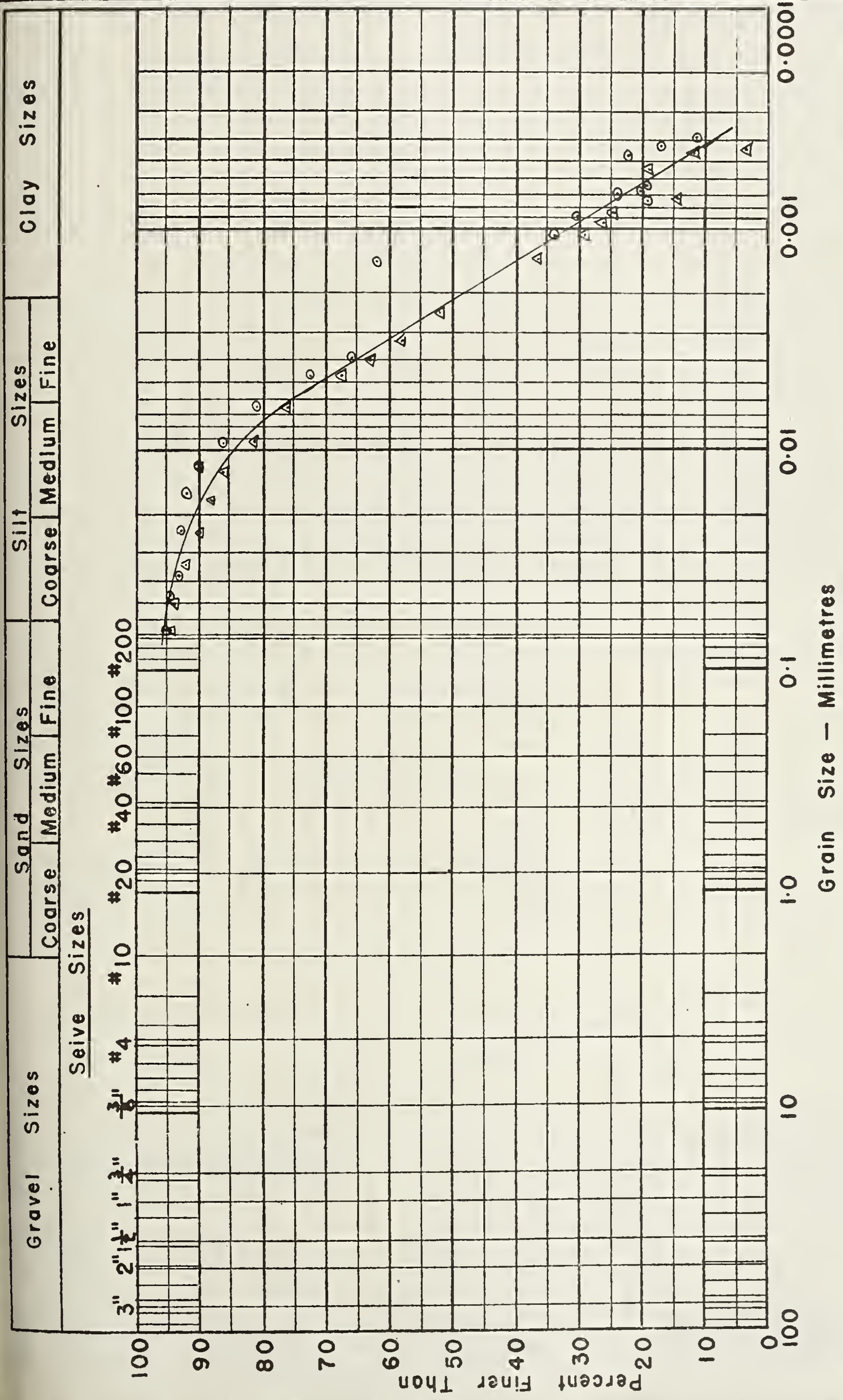
SOIL MECHANICS LABORATORY

GRAIN SIZE CURVE

PROJECT THESIS
SITE
SAMPLE CALCIUM (Group I)
LOCATION
HOLE DEPTH
TECHNICIAN ST & DW DATE 24 MAY '61



Legend
Sample 1 ⊙
Sample 2 △

D_{10} = ____ mm.
D_{60} = ____ mm.
C_u ____

Note: M.I.T. Grain Size Scale

Remarks: From work of Hamilton
W_L = 79.7
W_p = 31.0
I_p = 48.7
Compressive Index 0.68

PLATE H2

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HYDROMETER TEST

PROJECT THESIS
SITE
SAMPLE CALCIUM No1 (GP.1)
LOCATION
HOLE DEPTH
TECHNICIAN DW DATE 16 MAY 61

Date	Temp·	Time	Elapsed Time	R'_h	$R_h = R'_h + c_m$	D m.m.	$R_h + m_t - c_d$	W·%	W % Basis Orig. Sample	Remarks
16May61		1340	0							
			·5 MIN	19.7	20.2	·069	20.3	95.0		
			1	19.6	20.1	·049	20.2	94.5		
			2	19.4	19.9	·039	20.0	93.6		
			4	19.4	19.9	·024	20.0	93.6		
	22.5		8	19.2	19.7	·017	19.8	92.6		
			15	18.8	19.3	·013	19.4	90.7		
	22.3		30	17.9	18.4	·0093	18.5	86.5		
	22.1		1hr	16.8	17.3	·0066	17.4	81.4		
	21.6		2	15.1	15.6	·0048	15.6	73.0		
	21.2		3	13.8	14.3	·0040	14.2	66.4		
7May	20.6	0840	19:00	18.7	19.2	·0015	19.0	88.9		?
✓	20.7	1650	27:10	13.0	13.5	·0013	13.3	62.2		?
8May	20.3	0935	43:55	7.0 4.8	7.5 5.3	0011 ·0007	7.3 5.1	34.2 23.9		
9May	21.2	0915	67:35	6.1 4.5	6.6 5.0	·0009 ·00056	6.5 4.9	30.4 22.9		
0 ✓	21.4	1000	92:20	5.3 4.3	6.0 4.8	00076 ·00048	6.0 4.8	28.1 22.5		
1 ✓	23.4	20.45	127:05	3.3 2.7	3.8 3.2	00065 ·00043	4.2 3.6	19.7 16.9		
22	22.8	2015	150:35	1.7	2.2	·0004	2.4	11.2		

Hydrometer No.s 9857R and 50700 R Graduate No.

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s-1}(R_h + m_t - c_d) =$ 4.68 $(R_h + m_t - c_d)$

Meniscus correction = c_m = 0.5 and 0.5 respectively

Dispersing agent used Calgon Amount 10ml

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = 0.3 and 0.3 respectively

Specific Gravity of Solids = G_s 2.73

Description of Sample

Method of Preparation

Remarks

Initial Moisture Content

Container No.
Wt. Sample Wet + Tare
Wt. Sample Dry + Tare
Wt. Water
Tare Container
Wt. of Dry Soil
Initial Moisture w %

Dry Weight of Sample

Container No. 3
Wt. Sample (~~Wet~~/Dry) + Tare 773.51
Tare 739.82
Wt. (~~Wet~~/Dry) Soil 33.69
Dry Weight from Initial Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist.}\%}$ =

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HYDROMETER TEST

PROJECT *THESIS*
SITE
SAMPLE *Calcium No 2 (Gp 1)*
LOCATION
HOLE DEPTH
TECHNICIAN *DW* DATE *17May 61*

Date	Temp.	Time	Elapsed Time	R'_h	$R_h = R'_h + c_m$	D m.m.	$R_h + m_t - c_d$	W.%	W % Basis Orig. Sample	Remarks
17MAY		0915	0							
			.5 min	20.4	20.9	.069	21.0	94.5		
			1	20.2	20.7	.050	20.8	93.5		
			2	19.9	20.4	.035	20.5	92.1		
			4	19.4	19.9	.025	20.0	90.0		
	21.9		8	19.0	19.5	.018	19.6	88.1		
			15	18.6	19.1	.013	19.2	86.3		
	21.8		30	17.6	18.1	.0095	18.2	81.8		
	21.6		1 hr	16.5	17.0	.0066	17.0	76.5		
	21.3		2	14.6	15.1	.0048	15.0	67.5		
	21.1		3	13.7	14.2	.0040	14.1	63.4		
	21.0		4:24	12.5	13.0	.0030	12.9	58.0		
	20.8	1650	7:35	11.2	11.7	.0025	11.6	52.1		
18May	20.3	0930	24:15	7.9 5.6	8.4 6.1	.0015 .00095	8.1 5.8	36.4 26.1		
19 ✓	21.2	0915	48:00	6.2 4.2	6.7 4.7	0011 00068	6.6 4.6	29.6 20.7		
20 ✓	21.4	1000	72:45	5.0 3.8	5.5 4.3	00087 .00055	5.5 4.3	24.7 19.3		
21 ✓	23.5	2045	107:30	2.2 1.6	2.7 2.1	00072 .00047	3.1 2.5	13.9 11.2		
22 ✓	22.9	2015	131:00	0.8	1.3	.00045	1.6	3.56		

Hydrometer No.s *9857R* and *50700R* Graduate No.

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s - 1} (R_h + m_t - c_d) =$ *4.5* $(R_h + m_t - c_d)$

Meniscus correction = c_m = *0.5* and *0.5* respectively

Dispersing agent used *Calgon* Amount *10 ml.*

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = *0.3* and *0.3* respectively

Specific Gravity of Solids = G_s *2.73*

Description of Sample

Method of Preparation

Remarks

Initial Moisture Content

Container No.
Wt. Sample Wet + Tare
Wt. Sample Dry + Tare
Wt. Water
Tare Container
Wt. of Dry Soil
Initial Moisture *w* %

Dry Weight of Sample

Container No. *DK*
Wt. Sample (~~Wet~~/Dry) + Tare *764.77*
Tare *729.68*
Wt. (~~Wet~~/Dry) Soil *35.09*
Dry Weight from Initial Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist. \%}}$ =

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SPECIFIC GRAVITY

PROJECT THESIS
SITE
SAMPLE CALCIUM (Gp 1)
LOCATION
HOLE DEPTH
TECHNICIAN ST & DW DATE 16 May 61

Sample No.		
Flask No.	101	101
Method of Air Removal		
W_{b+w+s}	688.44	688.83
Temperature T	23.3°C	21.7°C
W_{b+w}	667.54	667.72
Evaporating Dish No.		
Wt. Sample Dry + Dish		
Tare Dish		
W_s	32.86	33.33
G_s	2.73	2.73

W_{b+w+s} = Weight of flask + water + sample at T°.
W_{b+w} = Weight of flask + water at T° (flask calibration curve).
W_s = Weight of dry soil
G_s = Specific gravity of soil particles = $\frac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.	(Air dry)		Sample No.		
Container No.	A-18		Container No.		
Wt. Sample Wet + Tare	73.4563		Wt. Test Sample Wet + Tare		
Wt. Sample Dry + Tare	73.2428		Tare Container		
Wt. Water	0.2135		Wt. Test Sample Wet		
Tare Container	66.6165		W_s		
Wt. of Dry Soil	6.6263				
Moisture Content w %	3.22				

Description of Sample:
wt bottle + Soil = 203.15 203.43
wt bottle = 169.23 169.23
Wt Soil (Air dry) = 33.92 34.20
Wt soil oven dry = 33.92/1.0322 = 32.86 34.20/1.0322 = 33.33

Remarks:

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Department of Civil Engineering
Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Calcium (Gp 1)
Depth sample 1
Engineer ST
Technician DW
Date of Set-up 21 May 61
Test Lateral Pressure 1.50 Kg/cm²

ple Description Average Cation Analysis:
h. Capacity 32.8 me./100gm Soil;
93.7%, Na 0.6%, K 2.1%; Mg 3.6%
Content 21.8 me/100gm soil
Wool (internal) Wicks
ial Weight 138.760 gms

h, mm	1. 80.42	2. 80.36	Aver: 80.39	Area Top	A_T 10.35	sq.mm
mm Top	1. 36.18	2. 36.44	Aver: 36.31	Area Centre	A_C 10.34	sq.mm
Centre	1. 36.28	2. 36.30	Aver: 36.29	Area Bottom	A_B 10.21	sq.mm
Bottom	1. 36.00	2. 36.10	Aver: 36.05	Average X-Sect Area		

al Volume 82.88 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 10.31$

OLIDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
1335	0	23.10	
	.1	22.80	
	.25	22.72	
	.50	22.65	
	1	22.50	
	2	22.30	
	4	21.95	
	6	21.65	
	8	21.40	
	10	21.20	
	15	20.70	
	20	20.30	
	30	19.70	
	65	18.10	
1620	165	15.60	
0800	1105	13.15	
1505	1530	13.00	
0800	2545	12.93	
0805	3990	12.82	
0800	5425	12.75	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
8 May 0800	8305	12.80	10.30

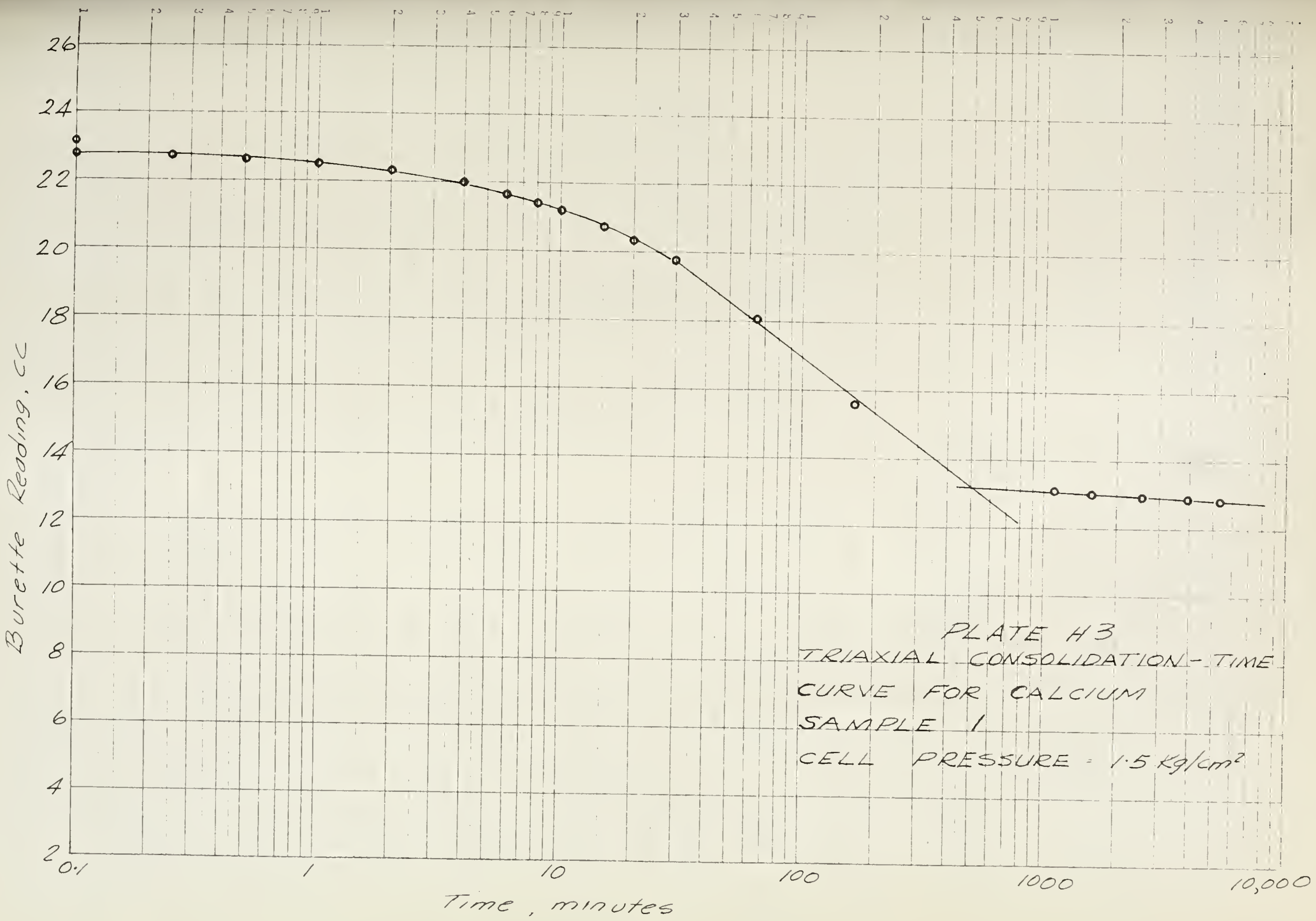

PLATE H3
TRIAXIAL CONSOLIDATION - TIME
CURVE FOR CALCIUM
SAMPLE 1
CELL PRESSURE = 1·5 Kg/cm²

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Department of Civil Engineering

Soil Mechanics Laboratory

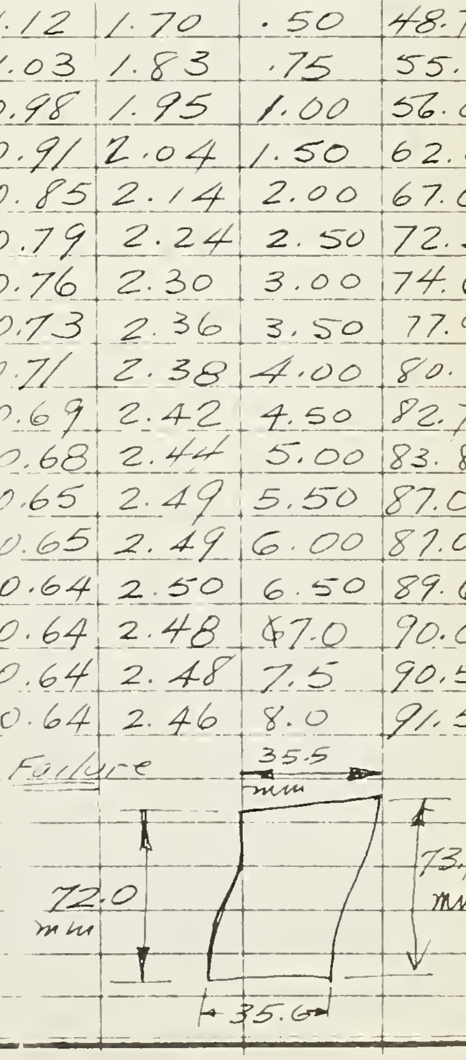
L COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Calcium
Depth Sample 1
Engineer S.T.
Technician D.N.
Date of Test 9 May 61
Test Lateral Pressure σ_3 1.5
Back Pressure 2.0
Remarks
Rate of Strain 600 min/10 mm
Proving Ring No 3-1498
Area Correction Factor 1.097

Failure
σ_3) = 0.96 σ_1 = 2.46
0.86
1.60
0.64
6.5 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$
0	10.31	0		0	-0.07					
2	10.31	20	.0525	0.11	-.01				.024	
3	10.31	30	.0517	0.16	0.0				0.037	
8	10.32	60	.0501	0.32	+.07	1.75	1.43	1.22	.10	21.9
20	10.33	115	.0481	0.59	+.22	1.87	1.28	1.46	.25	37.2
40	10.36	157	.0470	0.78	.38	1.90	1.12	1.70	.50	48.7
60	10.39	173	.0467	0.85	.47	1.88	1.03	1.83	.75	55.3
80	10.41	182	.0465	0.93	.52	1.91	0.98	1.95	1.00	56.0
120	10.46	195	.0464	0.95	.59	1.86	0.91	2.04	1.50	62.0
160	10.51	201	.0463	0.97	.65	1.82	0.85	2.14	2.00	67.0
200	10.57	205	.0462	0.98	.71	1.77	0.79	2.24	2.50	72.5
241	10.62	207	.0462	0.99	.74	1.75	0.76	2.30	3.00	74.6
281	10.67	208	.0462	0.99	.77	1.72	0.73	2.36	3.50	77.9
321	10.72	208	.0462	0.98	.79	1.69	0.71	2.38	4.00	80.6
361	10.78	209	.0461	0.98	.81	1.67	0.69	2.42	4.50	82.7
401	10.85	210	.0461	0.98	.82	1.66	0.68	2.44	5.00	83.8
441	10.89	210	.0461	0.97^5	.85	1.62	0.65	2.49	5.50	87.0
482	10.95	210	.0461	0.97	.85	1.62	0.65	2.49	6.00	87.0
521	11.01	209	.0461	0.96	.86	1.60	0.64	2.50	6.50	89.6
561	11.06	209	.0461	0.95^5	.86	1.59	0.64	2.48	7.0	90.0
601	11.12	209	.0461	0.95	.86	1.59	0.64	2.48	7.5	90.5
642	11.19	208	.0461	0.94	.86	1.58	0.64	2.46	8.0	91.5

NITIAL Moisture Content 56.21
INAL Moisture Content = 46.25
NITIAL Degree of Saturation 99.2
FINAL ✓ ✓ ✓ 100 (assumed)
INITIAL VOID Ratio 1.547
FINAL ✓ ✓ 1.262 (e = m.c × Gs)
PORE PRESSURE REACTION 96%



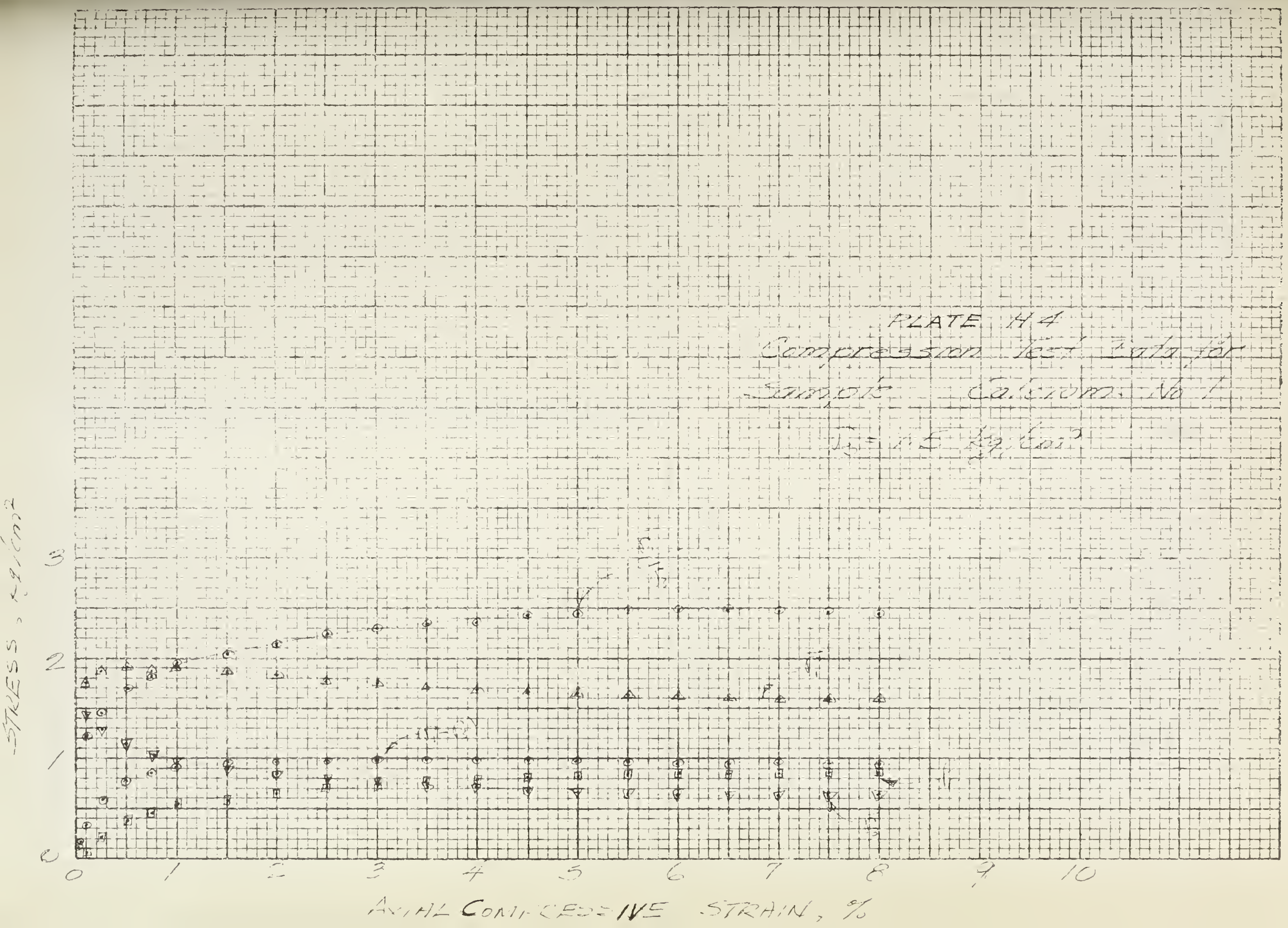


PLATE H4

Compression Test Data for Sample Calcium No 1

σ₃ = 1.5 kg/cm²

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Department of Civil Engineering
Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Calcium Gp 1
Depth sample 2
Engineer ST
Technician DW
Date of Set-up 4 May 61
Test Lateral Pressure 2.75 kg/cm²

mple Description Gp 1 cation analysis
ee data sheets for Ca #1

5 Wool Wicks
tial Weight 153.880 gms

th, mm	1. 80.54	2. 80.46	Aver: 80.50	Area Top	A_T 9.770	sq.mm
. mm Top	1. 35.20	2. 35.34	Aver. 35.27	Area Centre	A_C 9.965	sq.mm
Centre	1. 35.66	2. 35.58	Aver: 35.62	Area Bottom	A_B 10.004	sq.mm
Bottom	1. 35.68	2. 35.70	Aver: 35.69	Average X-Sect Area		

ial Volume 79.90 cc

$= ¼ (A_T + 2 A_C + A_B)$

$= 9.93$

OLIDATION DATA

	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
1145	0	24.00	
	·1	23.40	
	·25	23.25	
	·50	23.00	
	1.0	22.60	
	2.0	22.10	
	3.0	21.60	
	4.0	21.10	
	6.0	20.50	
	8.0	19.90	
	10.0	19.30	
	15	18.20	
	21	17.10	
	31	15.60	
	40	14.60	
	70	12.60	
	100	11.10	
1510	205	10.40	
700	315	10.15	
355	730	9.90	

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
5 May 0805	1220	9.85	
1550	1685	9.80	
6 May 0800	2655	9.80	
8 " 0800	5535	9.90	14.10
			(K = 1.143

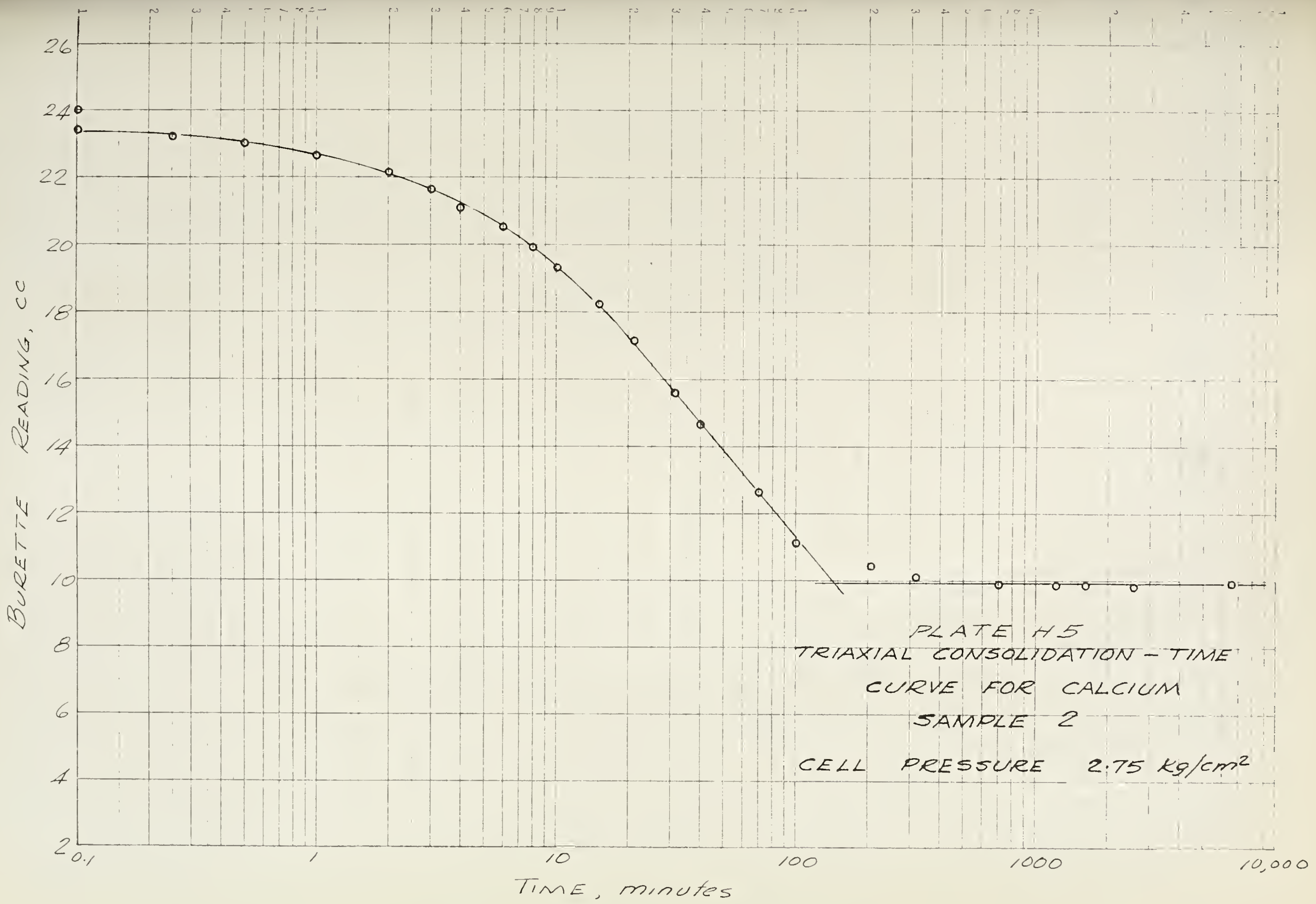
PLATE H5
TRIAXIAL CONSOLIDATION - TIME
CURVE FOR CALCIUM
SAMPLE 2

CELL PRESSURE 2.75 kg/cm²

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Department of Civil Engineering

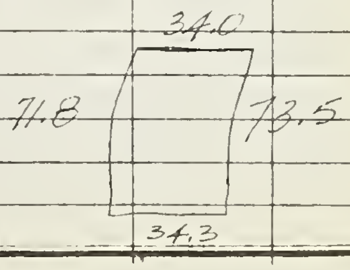
Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Calcium
Depth Sample 2
Engineer ST
Technician DW
Date of Test 11 May 60
Test Lateral Pressure σ_3 2.75 Kg/cm²
Back Pressure 2.0 Kg/cm²
Remarks
Rate of Strain 600 min/10 mm
Proving Ring 1498
Area Correction Factor 1.143

Failure
σ_3) = 1.79 σ_1= 4.54
1.66
2.88
1.09
8.50 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$ %
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
0	9.93	0		0	+.03					
6	9.93	20	.0525	0.12	+.08	2.79	2.67	1.04	.07	66.7
7	9.93	30	.0517	0.18	.11	2.82	2.64	1.07	.09	61.6
8	9.94	40	.0511	0.23	.12	2.86	2.63	1.09	.10	52.2
20	9.95	178	.0466	.95	.43	3.27	2.32	1.41	.25	45.3
40	9.98	251	.0456	1.31	.67	3.39	2.08	1.63	.50	51.1
60	10.01	280	.0453	1.45	.81	3.39	1.94	1.75	.75	55.9
80	10.04	296	.0451	1.52	.95	3.32	1.80	1.85	1.0	62.5
120	10.08	318	.0449	1.62	1.08	3.29	1.67	1.97	1.5	66.5
161	10.13	333	.0447	1.68	1.20	3.23	1.55	2.08	2.0	71.4
201	10.19	344	.0446	1.73	1.28	3.20	1.47	2.18	2.5	73.9
241	10.24	352	.0446	1.76	1.33	3.18	1.42	2.24	3.0	75.5
281	10.29	358	.0445	1.77	1.39	3.13	1.36	2.30	3.5	78.5
321	10.34	364	.0445	1.79	1.44	3.10	1.31	2.36	4.0	80.4
361	10.40	369	.0444	1.80	1.49	3.06	1.26	2.43	4.5	82.7
421	10.49	375	.0444	1.81	1.53	3.03	1.22	2.48	5.25	84.5
482	10.57	379	.0444	1.82	1.59	2.98	1.16	2.57	6.0	87.4
522	10.63	381	.0443	1.82	1.60	2.97	1.15	2.58	6.5	87.9
563	10.69	383	.0443	1.81	1.63	2.93	1.12	2.61	7.0	90.0
603	10.74	383	.0443	1.81	1.66	2.90	1.09	2.66	7.5	91.7
643	10.80	383	.0443	1.80	1.63	2.92	1.12	2.61	8.0	90.5
663	10.83	383	.0443	1.79	1.65	2.89	1.10	2.63	8.25	92.2
683	10.86	383	.0443	1.79	1.66	2.88	1.09	2.64	8.50	92.6
INAL MOISTURE	CONTENT	=		55.56%						
AL	✓	✓	=	40.93						
INAL Degree of	Saturation	=		98.9						
IAL	✓ ✓	✓	=	99.23						
GINAL ~~MOISTURE~~ VOID	RATIO ~~CONTENT~~	=		1.534						
IAL VOID RATIO			=	1.126						



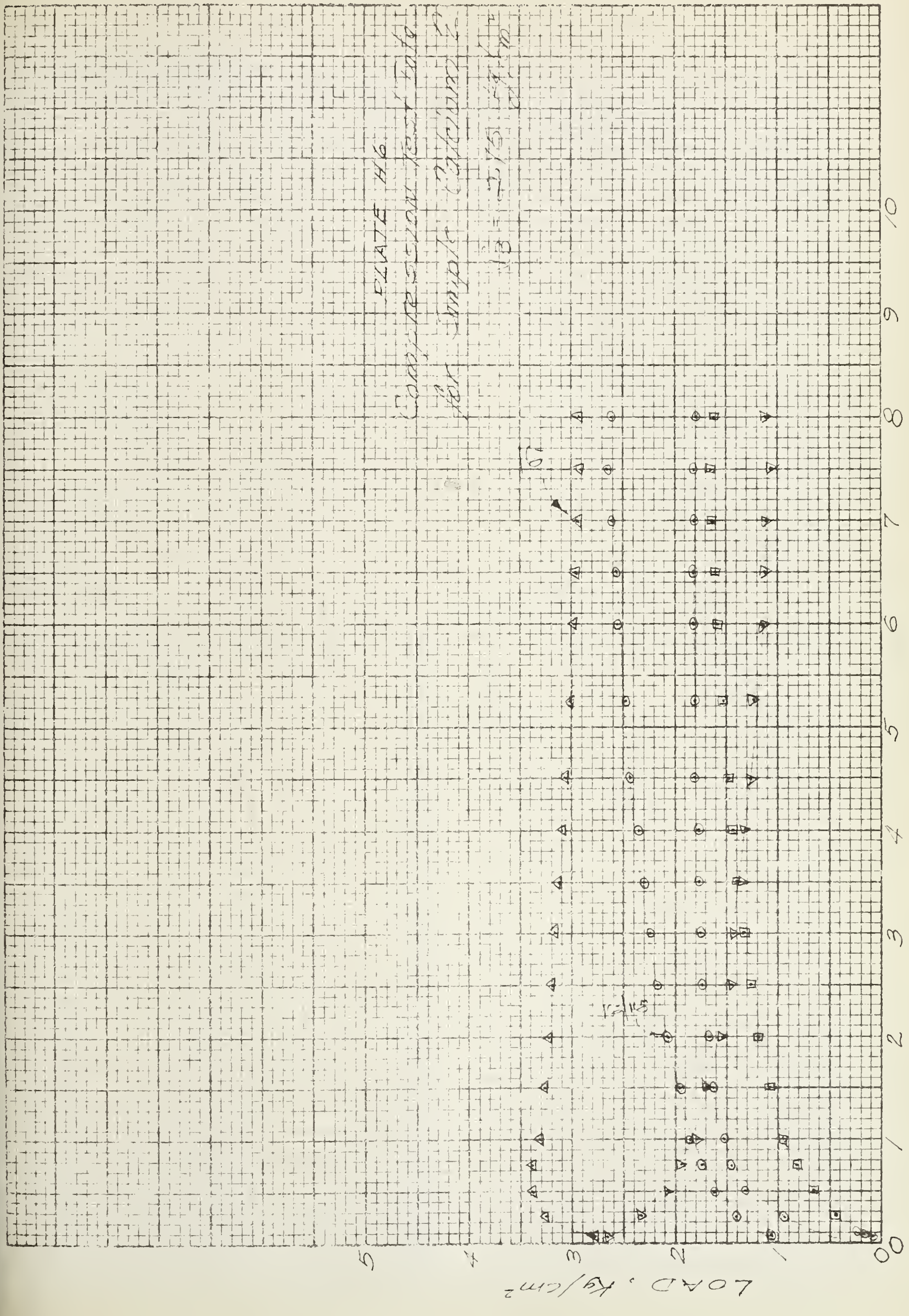


PLATE H6
Compression test data
for Sample Column 2
$\sigma_3 = 275$ g/cm^2

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Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

le Description For Cation Analysis See Data Sheets For Ca #1
ote:
is soil used by Hamilton
recheck W_L for his thesis
accounts for specimen being
orter than usual.
al Weight 122.041 gms

Project THESIS
Hole No. Calcium Gp 1
Depth
sample 3
Engineer ST
Technician
Date of Set-up 5 May 61
Test Lateral Pressure 4 Kg/cm^2

, mm	1. 73.00	2. 73.00	Aver: 73.00	Area Top	A_T 10.100	Sq.mm
mm Top	1. 35.80	2. 35.92	Aver: 35.86	Area Centre	A_C 9.892	sq.mm
Centre	1. 35.48	2. 35.50	Aver: 35.49	Area Bottom	A_B 9.776	sq.mm
Bottom	1. 35.30	2. 35.26	Aver: 35.28	Average X-Sect Area		

l Volume 72.40 cc

Average X-Sect Area $= \frac{1}{4}(A_T + 2A_C + A_B)$
$= 9.92$

LIDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
1330	0	24.30	
	.1	23.70	
	.25	23.55	
	.5	23.30	
	1	23.00	
	2	22.50	
	3	22.05	
	4	21.65	
	6	21.00	
	8.5	20.30	
	10	19.90	
	15	18.90	
	20	17.90	
	30	16.40	
	40	15.20	
	60	13.50	
	80	12.40	
545	135	11.30	
15	225	10.25	
335	605	9.70	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
6 May 0800	1110	9.62	
8 ✓ 0800	3990	9.62	
9 ✓ 1530	5880	9.62	
11 ✓ 1500	8720	9.60	14.70
			(K=1.15

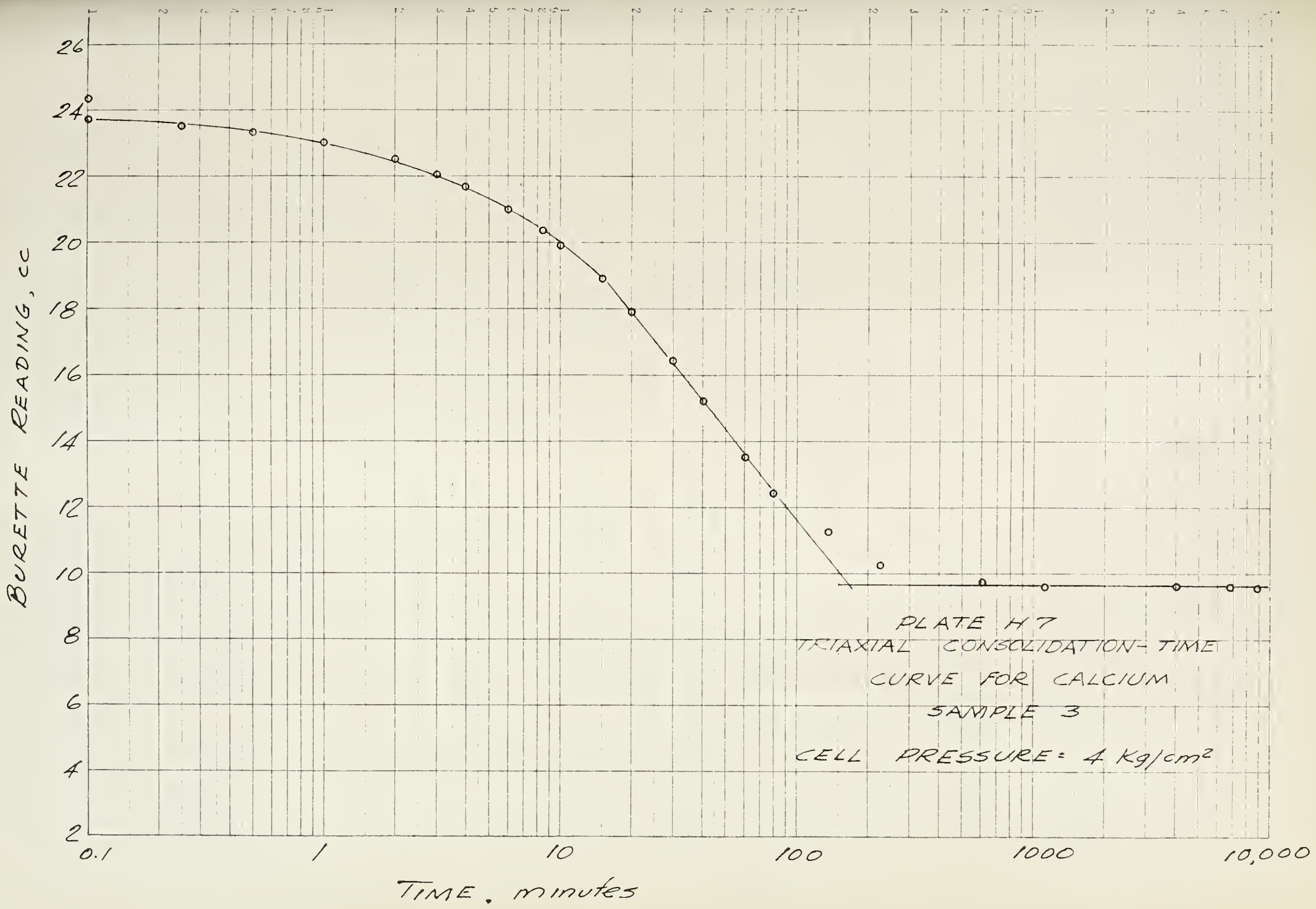

PLATE H7
TRIAXIAL CONSOLIDATION-TIME
CURVE FOR CALCIUM
SAMPLE 3

CELL PRESSURE: 4 Kg/cm²

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Department of Civil Engineering

Soil Mechanics Laboratory

L COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Calcium
Depth
Sample 3
Engineer ST
Technician
Date of Test 15 May 61
Test Lateral Pressure σ_3 4 kg/cm²
Back Pressure 2 ✓
Remarks
Rate of Strain 600 min/10mm
Proving Ring #3-1498
Area Correction Factor 1.15

Failure
σ_3) = 2.47 σ_1= 6.67
2.03
4.44
1.97
7.50 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\overline{\sigma_1}$ Major	Effective Stress $\overline{\sigma_3}$ Minor	Stress Ratio $\frac{\overline{\sigma_1}}{\overline{\sigma_3}}$	Axial Comp. Strain %	$\overline{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$ %
0	9.92			0	-.12					
3	9.92	20	.0525	.12	-.14	4.26	4.14	1.03	.04	
5	9.93	35	.0514	.21	-.12	4.35	4.12	1.06	.07	
8	9.93	97	.0487	.55	-.01	4.56	4.01	1.14	.11	
14.5	9.94	177	.04667	.95	+.23	4.72	3.77	1.25	.20	24.2
22	9.95	247	.04562	1.30	+.43	4.87	3.57	1.36	.30	33.1
29.5	9.96	299	.04503	1.55	+.58	4.97	3.42	1.45	.40	37.4
36.5	9.96	330	.04478	1.71	+.71	5.00	3.29	1.52	.50	41.5
44	9.97	353	.04458	1.81	.84	4.97	3.16	1.57	.60	46.4
51	9.98	367	.04448	1.88	.86	5.02	3.14	1.60	.70	45.7
58.5	9.99	380	.04436	1.94	.92	5.02	3.08	1.63	.80	47.4
66	10.00	389	.04430	1.98	.99	4.99	3.01	1.66	.90	50.0
73	10.01	396	.04425	2.01	1.04	4.97	2.96	1.68	1.00	51.7
87.5	10.03	409	.04416	2.07	1.10	4.97	2.90	1.71	1.20	53.1
102	10.05	421	.04406	2.12	1.20	4.92	2.80	1.76	1.40	56.6
117	10.07	428	.04401	2.15	1.24	4.91	2.76	1.78	1.60	57.7
146	10.10	443	.04392	2.22	1.39	4.83	2.61	1.85	2.00	62.6
164	10.13	451	.04389	2.25	1.43	4.82	2.57	1.88	2.25	63.6
182	10.15	459	.04383	2.28	1.52	4.76	2.48	1.92	2.50	61.3
209	10.19	466	.04380	2.30	1.60	4.70	2.40	1.96	2.86	69.6
222	10.20	472	.04377	2.33	1.64	4.69	2.36	1.99	3.04	70.0
237	10.22	476	.04373	2.34	1.63	4.71	2.37	1.99	3.25	69.7
255	10.25	481	.04370	2.36	1.71	4.65	2.29	2.03	3.50	72.5
274	10.27	485.5	.04369	2.38	1.71	4.67	2.29	2.04	3.15	71.8
292	10.29	490	.04365	2.39	1.73	4.66	2.27	2.05	4.00	74.1
310	10.32	492	.04365	2.39	1.75	4.64	2.25	2.06	4.25	73.2
328.5	10.35	497	.04360	2.41	1.80	4.61	2.20	2.10	4.50	74.7
347	10.37	502	.04360	2.43	1.84	4.59	2.16	2.13	4.75	75.7
365	10.40	506	.04358	2.44	1.89	4.55	2.11	2.16	5.00	77.5

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Sheet 2

H17

Project THESIS

Hole No. Calcium

Depth Sample 3

Area Correction Factor 1.15

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg Div	$\sigma_1 - \sigma_3$ $= \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg cm² P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ P_p / $\sigma_1-\sigma_3$ %
385	10.42	508	.04356	2.44	1.85	4.59	2.15	2.13	5.27	75.8
401.5	10.45	512	.04352	2.45	1.89	4.56	2.11	2.16	5.50	77.2
420	10.47	515	.04351	2.46	1.91	4.55	2.09	2.18	5.75	77.6
438	10.50	518	.04350	2.47	1.93	4.54	2.07	2.19	6.00	78.1
456	10.52	520	.04350	2.47	1.95	4.52	2.05	2.20	6.25	78.9
474.5	10.55	523	.04348	2.48	1.98	4.50	2.02	2.23	6.50	79.8
493	10.57	524.5	.04347	2.48	1.96	4.52	2.04	2.22	6.75	79.0
511	10.60	527	.04346	2.48	2.00	4.48	2.00	2.24	7.00	80.6
529	10.63	528.5	.04345	2.48	2.01	4.47	1.99	2.25	7.25	78.6
547.5	10.65	531	.04343	2.47	2.03	4.44	1.97	2.26	7.50	81.5
566	10.68	533.5	.04342	2.47	2.03	4.44	1.97	2.26	7.75	81.5
584	10.71	534	.04342	2.47	2.01	4.46	1.99	2.25	8.00	80.7
602	10.73	537	.04342	2.48 ‡	2.03	4.45	1.97	2.26	8.25	81.2

Pronounced buckling of specimen, piston rubbing on top cap.

Content – Initial 53.33 %

Final 37.63

Saturation – Initial 98.2 %

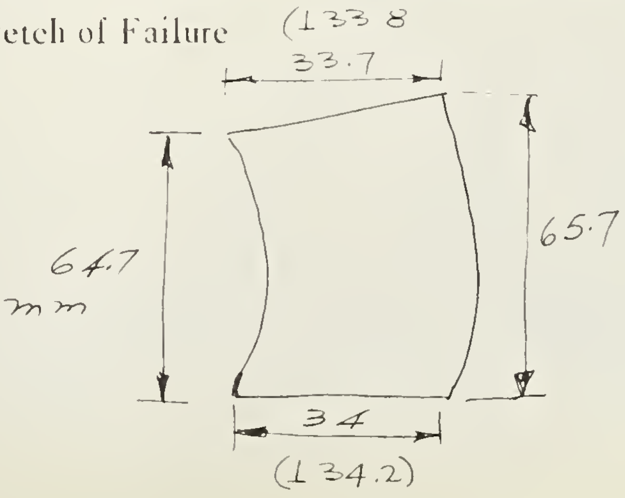
Final 99.3

– Initial 1.484

Final 1.034

sure Reaction 85 %

Sketch of Failure



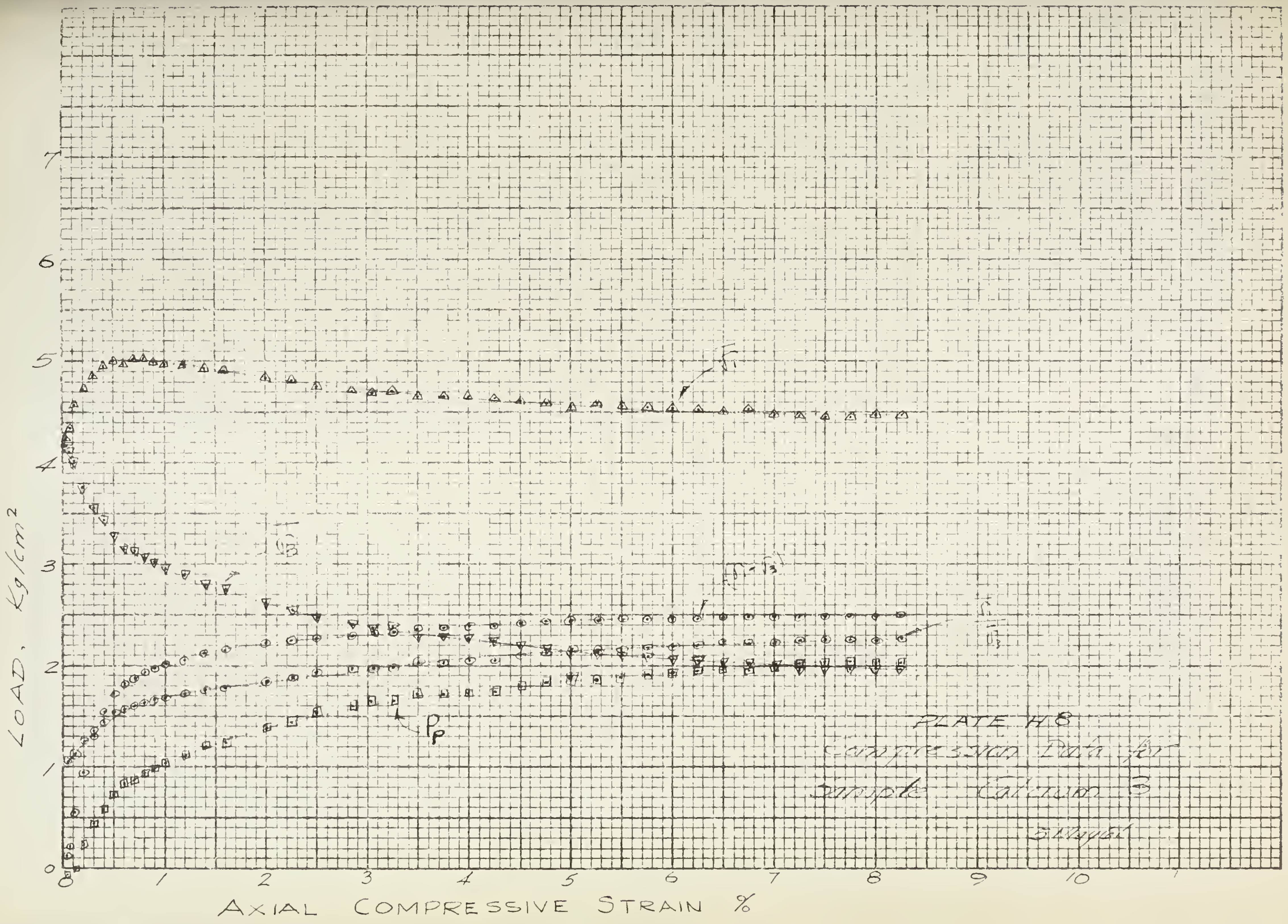


PLATE H 8
Compression Data for
Sample Calcium 3

5 Aug 51

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

nple Description Average Cation Analysis
ch Capacity 32·8 m·e/100gm Soil;
95·6% Na 0·7%, K 2.5%, Mg 1·2%; Salt
ontent 9·8 m.e./100gm soil
v 5 Wool Wicks
ial Weight 134·214 gms

Project THESIS
Hole No. Calcium (Gp 2)
Depth sample 4
Engineer ST
Technician DW
Date of Set-up 17 July 68
Test Lateral Pressure 5·25

h, mm	1. 80·60	2. 80·66	Aver: 80·63	Area Top	A_T 9·931 sq.mm
mm Top	1. 35·52	2. 35·60	Aver: 35·56	Area Centre	A_C 10·060 sq.mm
Centre	1. 35·78	2. 35·80	Aver: 35·79	Area Bottom	A_B 10·122 sq.mm
Bottom	1. 35·94	2. 35·86	Aver: 35·90	Average X–Sect Area	

al Volume 80·95 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 10·04$

OLIDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
1130	0	24·90	
	·1	23·90	
	·25	23·60	
	·5	23·30	
	1·0	22·80	
	2	21·80	
	3	21·15	
	4	20·60	
	6	19·50	
	8	18·~~8~~70	
	10	17·90	
	15	16·30	
	20	15·00	
	30	13·10	
	100	8·00	
1330	240	6·40	
1705	335	6·20	
830	1260	5·97	18·93

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.

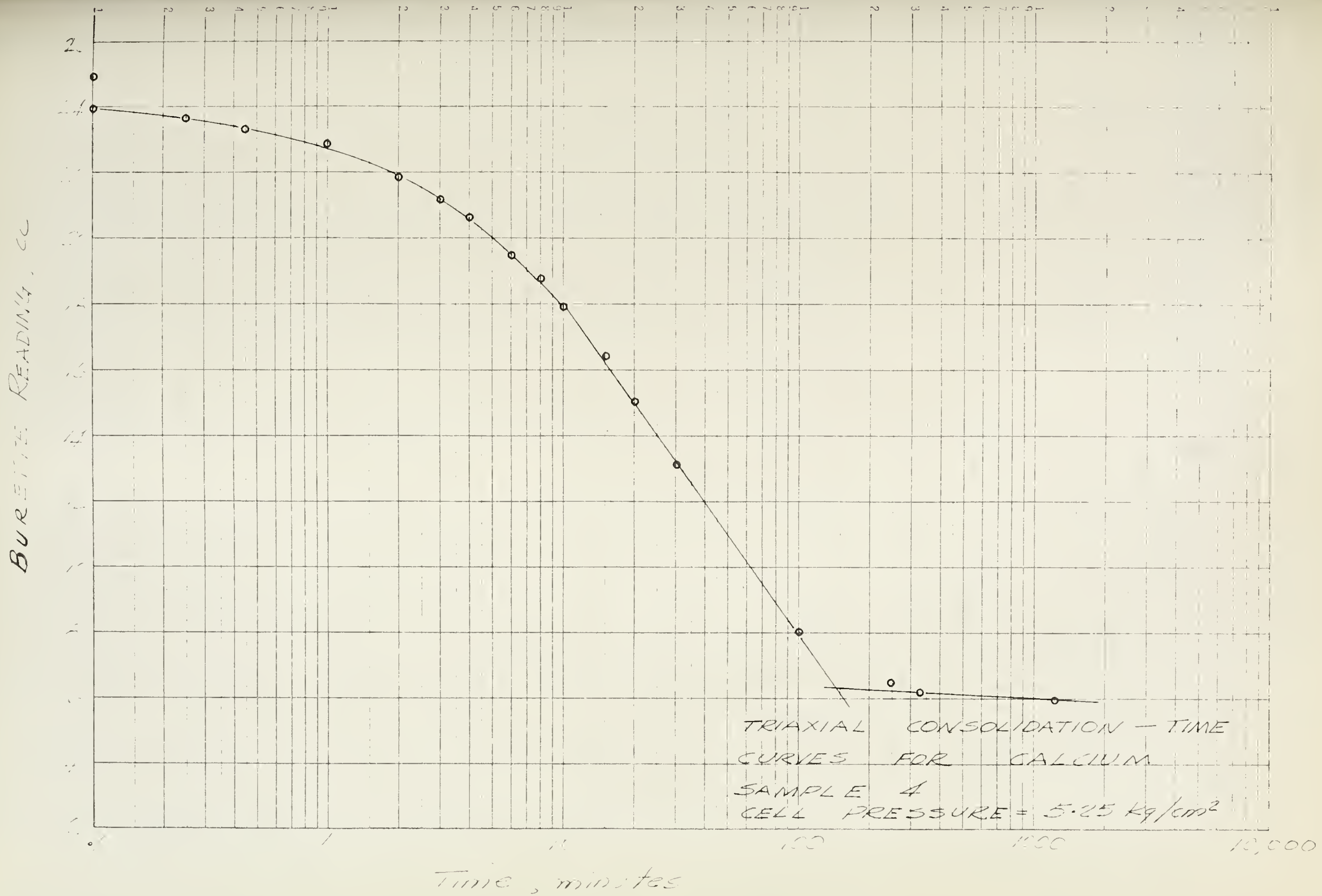

TRIAXIAL CONSOLIDATION — TIME
CURVES FOR CALCIUM
SAMPLE 4
CELL PRESSURE = 5·25 kg/cm²
BURETTE READING, cc
2.
1.4
Time, minutes
.1
1
10
100
1000
10,000

PLATE H9

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Failure

σ_3) = 3.33 σ_1= 8.58

3.07

5.51

2.18

. 11.0 %

Project THESIS

Hole No. Calcium

Depth Sample 4

Engineer ST

Technician DW

Date of Test 19 July 61

Test Lateral Pressure σ_3 5.25

Back Pressure 2.0

Remarks Proving Ring 3 - 1498 (50 kg)
Rate of strain 551 min/10mm

Area Correction Factor 1.20

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$ %
0	10.04	0			+.1					
5	10.04	20	.0525	0.1	+.14	5.21	5.11	1.02	.06	
10.5	10.05	40	.0511	0.24	0.18	5.31	5.07	1.05	.13	
16	10.06	54	.0504	.32	0.23	5.34	5.02	1.06	.20	
24	10.07	173	.04675	.96	0.50	5.71	4.75	1.20	.3	52.1
32	10.08	283	.04525	1.52	.78	5.99	4.47	1.34	.4	51.3
40.5	10.09	350	.04660	1.85	.97	6.13	4.28	1.43	.5	52.4
48.5	10.10	386	.04431	2.03	1.10	6.18	4.15	1.49	.6	54.2
56.5	10.11	412	.04413	2.16	1.21	6.20	4.04	1.53	.7	56.0
64.5	10.12	432	.0440	2.25	1.31	6.19	3.94	1.57	.8	58.2
73	10.13	448	.04390	2.33	1.40	6.18	3.85	1.61	.9	60.1
81	10.14	460	04383	2.39	1.48	6.16	3.77	1.63	1.00	61.9
89	10.15	471	.04377	2.44	1.54	6.15	3.71	1.66	1.10	63.1
97	10.16	481	.04370	2.48	1.60	6.13	3.65	1.68	1.20	64.5
105	10.17	490	.04366	2.52	1.65	6.12	3.60	1.70	1.30	65.5
113	10.18	497	.04361	2.55	1.70	6.10	3.55	1.72	1.40	66.7
121.5	10.19	505	.04357	2.59	1.75	6.09	3.50	1.74	1.50	67.6
129.5	10.20	512	.04354	2.62	1.78	6.09	3.47	1.76	1.60	67.9
137.5	10.21	519	.04350	2.65	1.84	6.06	3.41	1.78	1.70	69.4
154	10.235	533	.04342	2.71	1.96	6.00	3.29	1.82	1.90	72.3
162	10.245	539	.04340	2.74	2.00	5.99	3.25	1.84	2.0	73.0
178	10.27	550	.04335	2.79	2.13	5.91	3.12	1.89	2.2	76.3
194	10.29	560	04330	2.83	2.20	5.88	3.05	1.93	2.4	77.7
210.5	10.31	570	04327	2.87	2.29	5.83	2.96	1.97	2.6	79.8
227	10.33	577	.04323	2.90	2.35	5.80	2.90	2.00	2.8	81.0
243	10.35	586	.04320	2.94	2.39	5.80	2.86	2.03	3.0	81.3
283	10.41	605	.04311	3.01	2.49	5.77	2.76	2.09	3.5	82.7
324	10.46	620	.04307	3.06	2.59	5.72	2.66	2.15	4.0	84.6
364.5	10.515	636	.0430	3.12	2.68	5.69	2.57	2.21	4.5	85.9

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Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Calcium

Depth ______ Sample 4

Area Correction Factor 1.20

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1 - \sigma_3}$
405	10.575	654	.04293	3.19	2.72	5.72	2.53	2.26	5.0	85.3
445	10.63	662	.04290	3.21	2.79	5.67	2.46	2.30	5.5	86.9
485.5	10.685	670	.04288	3.23	2.83	5.65	2.42	2.33	6.0	87.6
526	10.74	679	.04283	3.25	2.88	5.62	2.37	2.37	6.5	88.6
566.5	10.80	686	.04281	3.26	2.91	5.60	2.34	2.39	7.0	89.3
607	10.86	694	.04279	3.28	2.96	5.57	2.29	2.43	7.5	90.2
647.5	10.92	700	.04277	3.29	2.96	5.58	2.29	2.44	8.0	90.0
688	10.975	707	.04273	3.30	2.99	5.56	2.26	2.46	8.5	90.6
728.5	11.04	713	.04271	3.31	3.01	5.55	2.24	2.48	9.0	90.9
890.5	11.29	736	.04262	3.33	3.07	5.51	2.18	2.53	11.0	92.2
931	11.355	737	.04262	3.32	3.07	5.50	2.18	2.52	11.5	92.5
971.5	11.42	740	.04261	3.31*	3.08	5.48	2.17	2.525	12.0	93.1
1012	11.475	742	.04260	3.31	3.08	5.48	2.17	2.525	12.5	93.1

Specimen noticeably bulged at centre, cap not tipped

Content – Initial 57.45%

Final 38.50

f Saturation – Initial 98.5

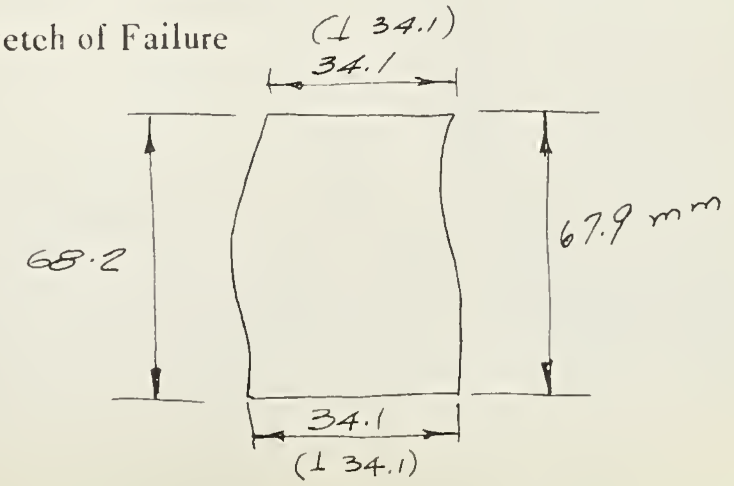
Final 98.1

io – Initial 1.59

Final 1.071

ssure Reaction 94%

Sketch of Failure



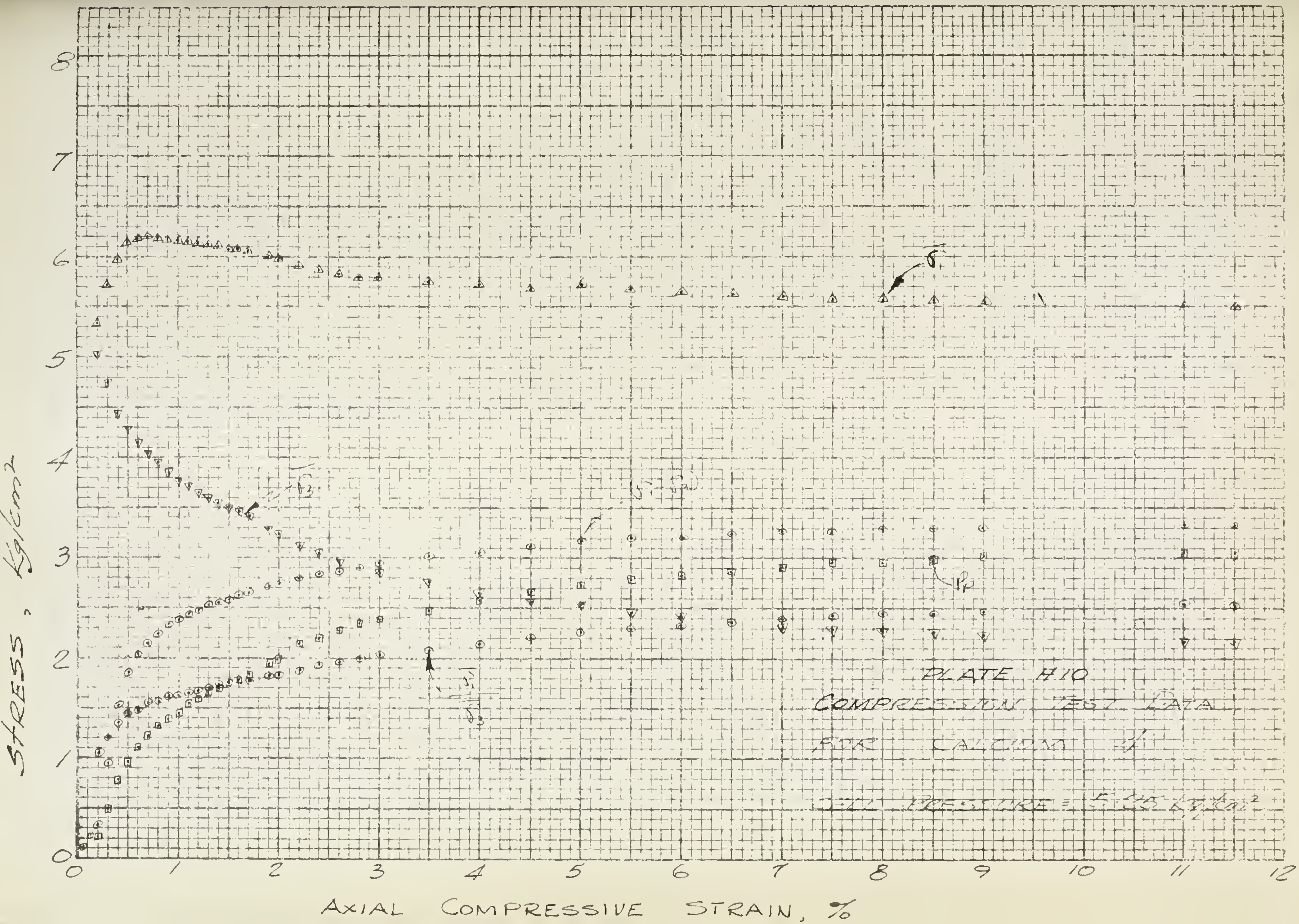


PLATE H10

COMPRESSION TEST DATA

FOR CALCIUM #4

CELL PRESSURE = 5.25 kg/cm²

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Soil Mechanics Laboratory

AXIAL COMPRESSION TEST ON COHESIVE SOIL

ple Description CEC 32·8 me/100gm soil
95·6, Na-0·7, K-2·5, Mg 1·2, S.C-9·8

al Weight 135·036 gms

Project THESIS

Hole No. Calcium Gp 2

Depth sample 5

Engineer ST

Technician Dvy

Date of Set-up 20 July '61

Test Lateral Pressure 6·5 kg/cm²

, mm	1. 80·36	2. 80·34	Aver: 80·35	Area Top A_T 10·355 sq.mm
mm Top	1. 36·38	2. 36·24	Aver: 36·31	Area Centre A_C 10·094 sq.mm
Centre	1. 35·98	2. 35·72	Aver: 35.85	Area Bottom A_B 9.859 sq.mm
Bottom	1. 35·50	2. 35·36	Aver: 35·43	Average X-Sect Area

l Volume 81·15 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

= 10.10

LIDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
040	0	24.90	
	1	24.20	
	.25	23.80	
	.5	23·30	
	1	22·60	
	2	21·50	
	3	20·60	
	4	19.80	
	5	19.10	
	6	18·50	
	8	17·40	
	10	16·50	
	15	14.65	
	20	13.25	
	25	12.20	
	30	11.30	
	40	10.05	
	50	9.15	
	85	7.55	
	160	6·50	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
~~24~~	200	6.33	
	300	6.15	
1720	400	6.10	
1905	505	6·03	
2040	600	6·00	18.90
			K=1.20

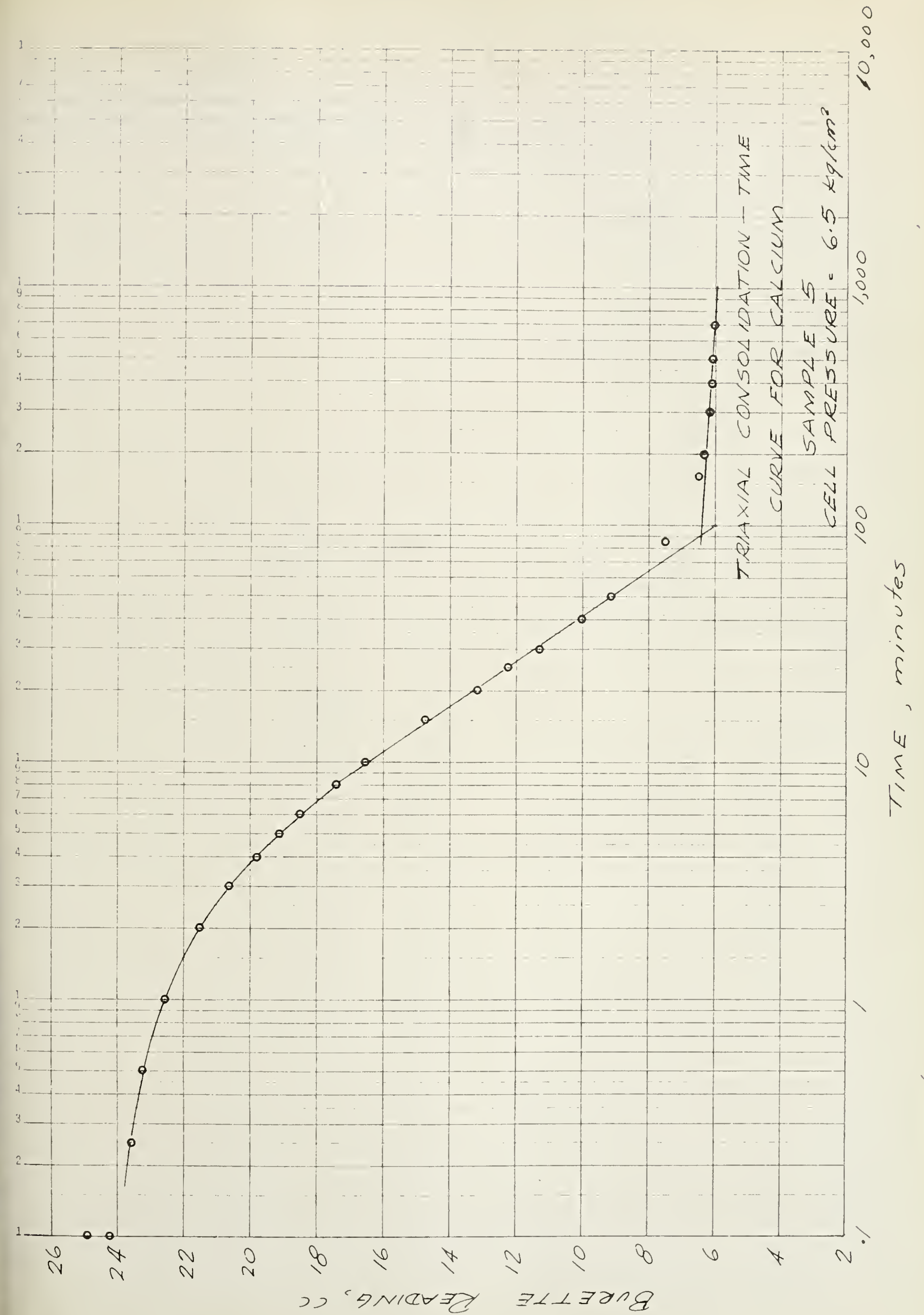
TRIAXIAL CONSOLIDATION – TIME
CURVE FOR CALCIUM
SAMPLE 5
CELL PRESSURE = 6.5 kg/cm²
TIME , minutes
.1
1
10
100
1,000
10,000
BURETTE READING, cc
2
4
6
8
10
12
14
16
18
20
22
24
26

PLATE H11

UNIVERSITY OF ALBERTA

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Soil Mechanics Laboratory

L COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Calcium

Depth Sample 5

Engineer ST

Technician DW

Date of Test 21 July 61

Test Lateral Pressure σ_3 6.5 kg/cm²

Back Pressure 2

Remarks Proving Ring 3-1498 (50kg)
~~Rate of Strain~~ 408 min /10mm

Area Correction Factor 1.20

Failure

σ_3) = 3.65 σ_1 = 10.15

3.66 kg/cm²

6.49

2.84

9.0 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}}{\frac{P_P}{\sigma_1 - \sigma_3}}$
0	10.10		0	0						
2	10.1	20	.0585	.0.12	.15				.025	
6.5	10.11	40	.0511	.24	.20				.08	
16	10.12	58	.0502	.35	.25				.20	71.4
24	10.13	76	.04945	.45	.30				.30	66.7
32	10.14	213	.0461	1.16	.68	6.98	5.82	1.20	.40	58.6
40	10.15	315	.0449	1.67	1.0	7.17	5.50	1.30	.50	59.9
48	10.16	373	.04442	1.96	1.17	7.29	5.33	1.37	.60	59.7
56	10.17	415	.04410	2.16	1.34	7.32	5.16	1.42	.70	62.0
64.5	10.185	448	.0439	2.32	1.48	7.34	5.02	1.46	.8	63.8
72.5	10.195	474	.04374	2.44	1.60	7.34	4.90	1.50	.9	65.6
80.5	10.205	495	.04363	2.54	1.70	7.34	4.80	1.53	1.0	66.9
88.5	10.215	512	.04353	2.62	1.80	7.32	4.70	1.56	1.1	68.7
96.5	10.225	527	.04346	2.69	1.85	7.34	4.65	1.58	1.2	68.8
104.5	10.235	541	.04340	2.75	1.94	7.31	4.56	1.60	1.3	70.5
112.5	10.25	553	.04335	2.81	2.02	7.29	4.48	1.63	1.4	71.9
120.5	10.26	563	.04330	2.85	2.09	7.26	4.41	1.65	1.5	73.3
128.5	10.27	574	.04324	2.90	2.15	7.25	4.35	1.67	1.6	74.1
136.5	10.28	584	.04320	2.94	2.21	7.23	4.29	1.69	1.7	75.2
144.5	10.29	593	.04317	2.99	2.28	7.21	4.22	1.71	1.8	76.3
152.5	10.30	603	.04313	3.03	2.34	7.19	4.16	1.73	1.9	77.2
160.5	10.31	610	.04310	3.06	2.40	7.16	4.10	1.75	2.0	78.4
177	10.33	627	.04303	3.13	2.51	7.12	3.99	1.78	2.2	80.1
193	10.35	641	.04299	3.19	2.60	7.09	3.90	1.82	2.4	81.5
209	10.375	654	.04293	3.25	2.69	7.06	3.81	1.85	2.6	82.8
225	10.395	665	.04289	3.29	2.74	7.05	3.76	1.88	2.8	83.3
241	10.415	675	.04285	3.33	2.79	7.04	3.71	1.90	3.0	83.7
281	10.47	700	.04277	3.43	2.96	6.97	3.54	1.97	3.5	86.3
321	10.52	721	.04268	3.51	3.10	6.91	3.40	2.03	4.0	88.3

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Calcium

Depth ______ Sample 5

Area Correction Factor 1·20

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_F	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$
321·5	10·58	740	.04261	3·58	3·22	6·86	3·28	2·09	4·5	90·0
402	10·635	754	·04257	3·62	3·28	6·84	3·22	2·12	5·0	90·6
442	10·69	766	.04252	3·66	3·36	6·80	3·14	2·17	5·5	91·8
485	10·75	777	·04250	3·69	3·44	6·75	3·06	2·21	6·0	93·2
522	10·80	785	.04247	3·70	3·45	6.75^{5}	3·05	2·21	6·5	93·2
562·5	10·86	790	.04245	3·71	3·52	6·69	2·98	2·25	7·0	94·9
603	10·92	794	·04242	3·70	3·54	6·66	2·96	2·25	7·5	95·6
643	10·98	797	.04241	369 ★	3·61	6·58	2·89	2·28	8·0	97·8
683	11·04	797	.04241	3·67	3·64	6·53	2·86	2·28	8·5	99·1
723	11·10	796	.04241	3·65	3·66	6·49	2·84	2·28	9·0	100·3
763	11·16	792	.04244	3·61	3·66	6·45	2·84	2·27	9·5	101·4
787·5	11·20	786	.04247	3·58	3·68	6·40	2·82	2·27	9·8	102·7
964	11·47	612	.04310	2·76	3·63	5·63	2·87	1·96	12·0	131·5

ecimen showing some distortion, flat "S"

ontent – Initial 57·79

Final 36·35

aturation – Initial 99·3

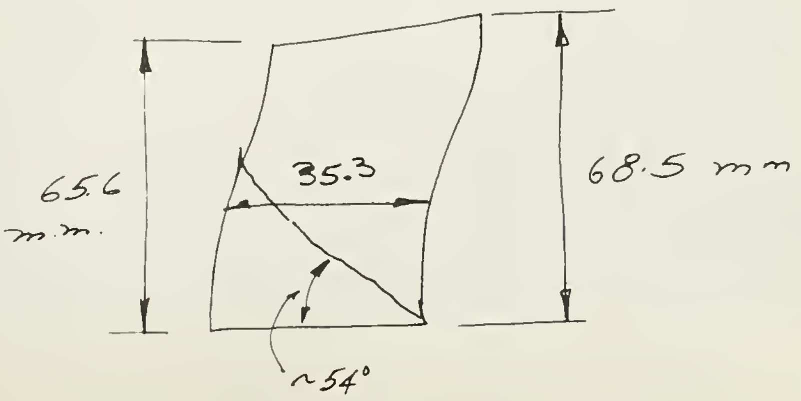
Final 98·6

– Initial 1·599

Final 1·007

ure Reaction 95%

Sketch of Failure



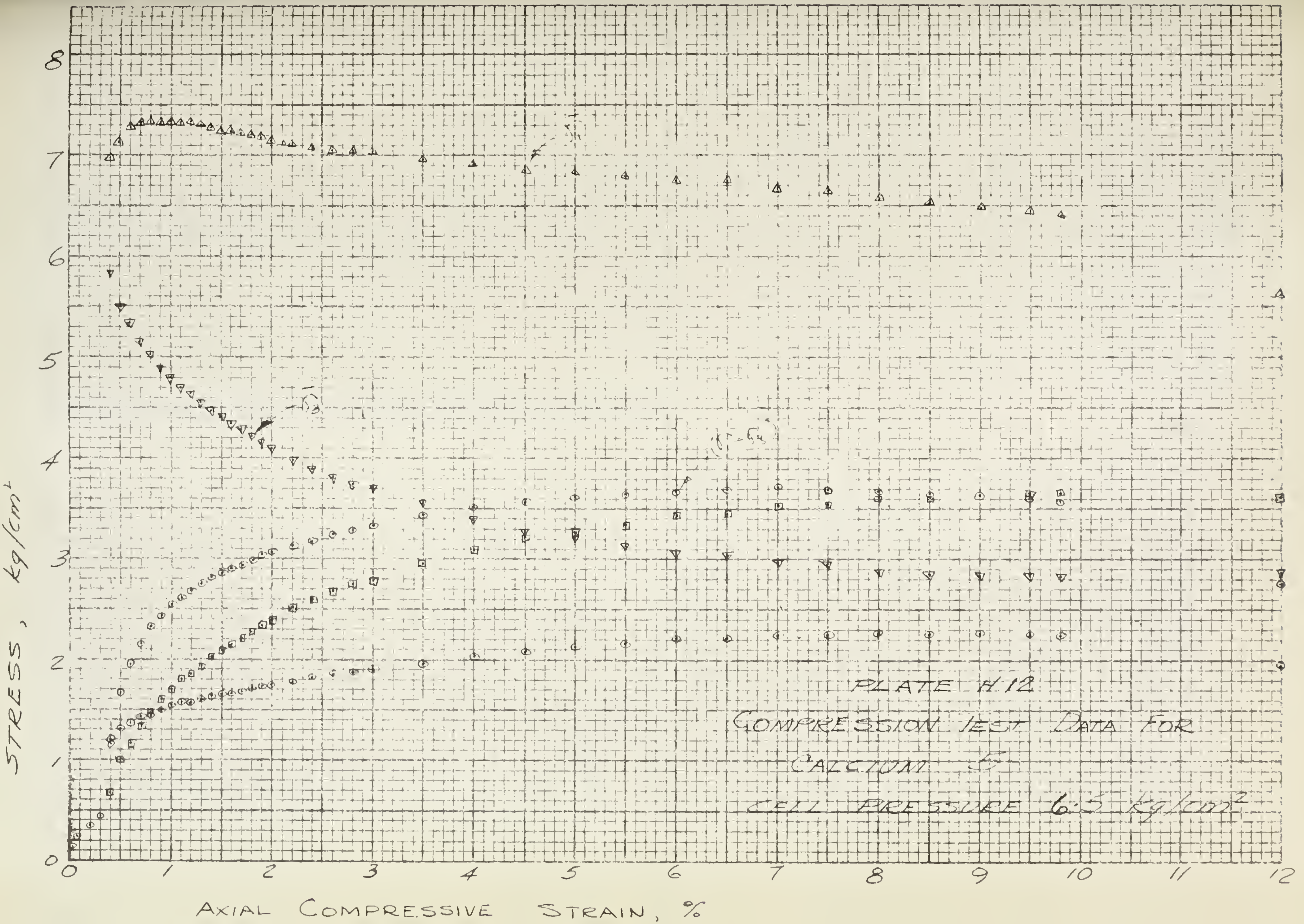


PLATE H12

COMPRESSION TEST DATA FOR

CALCIUM 5

CELL PRESSURE 6.5 kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Calcium Gp 2

Depth sample 6

Engineer ST

Technician DW

Date of Set-up 22 July 61

Test Lateral Pressure 4.0 kg/cm²

nple Description Repeat test of
mple #3 (σ_3=4) ; CEC 32.8, Ca 95.6
a 0.7, K 2.5, Mg 1.2, S.C. 9.8 m.e.

Wool (internal) wicks

ial Weight 132.531 gms

th, mm	1. 80.62	2. 80.60	Aver: 80.61
mm Top	1. 35.24	2. 35.32	Aver: 35.28
Centre	1. 35.52	2. 35.50	Aver: 35.51
Bottom	1. 35.68	2. 35.70	Aver: 35.69

Area Top A_T 9.776 sq.mm

Area Centre A_C 9.904 sq.mm

Area Bottom A_B 10.004 sq.mm

Average X–Sect Area

$= \frac{1}{4}(A_T + 2A_C + A_B)$

= 9.90

al Volume 79.80 cc

OLIDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
0950	0	24.70	
	.1	24.10	
	.25	23.80	
	.5	23.40	
	1	22.85	
	2	21.90	
	3	21.15	
	4	20.35	
	6	19.35	
	8	18.45	
	10	17.70	
	15	16.20	
	30	13.55	
	45	12.15	
	60	11.40	
	80	10.75	
	100	10.40	
	130	10.05	
1358	248	9.60	
530	340	9.50	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
2050	660	9.40	
23 July 1615	1825	9.32	
24 July 0810	2780	9.35	15.35
			(k=1.157)

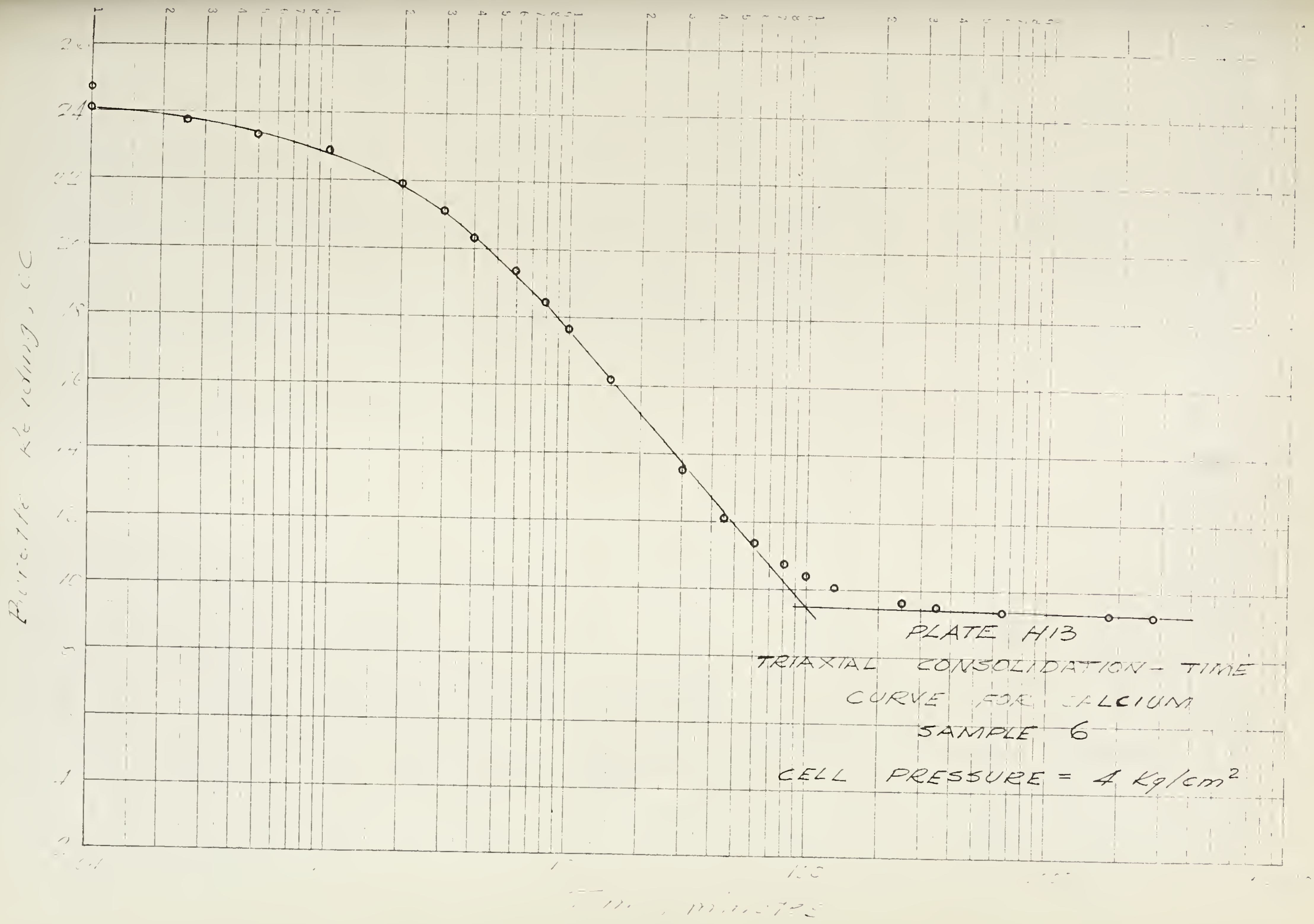


PLATE H13
TRIAXIAL CONSOLIDATION - TIME
CURVE FOR CALCIUM
SAMPLE 6
CELL PRESSURE = 4 Kg/cm^2

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

L COMPRESSION TEST ON COHESIVE SOIL

Failure

σ_3) = 2.58 σ_1= 6.58

2.45 kg/cm²

4.13

1.55

11.0 %

Project THESIS

Hole No. Calcium

Depth Sample 6

Engineer ST

Technician DW

Date of Test 25 July 61

Test Lateral Pressure σ_3 4.0 kg/cm²

Back Pressure 2

Remarks Proving Ring 3-1498
Rate of Strain 408 min/10mm

Area Correction Factor 1.157

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1-\sigma_3}$
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
0	9.90	0								
1	9.90	20	.0525	.12	+.08	4.04	3.92	1.03	.01	66.7
2	9.91	40	.0511	.24	+.12	4.12	3.88	1.06	.025	50.0
8	9.91	163	.04695	.89	.38	4.51	3.62	1.25	.1	42.7
16	9.92	249	.0456	1.32	.62	4.70	3.38	1.39	.2	46.9
24	9.93	298	.04507	1.56	.79	4.77	3.21	1.49	.3	50.6
32	9.94	327	.04480	1.70	.91	4.79	3.09	1.55	.4	53.5
40	9.95	349	.04461	1.81	1.04	4.77	2.96	1.61	.5	57.5
48.5	9.96	364	.0445	1.88	1.11	4.77	2.89	1.65	.6	59.0
56.5	9.97	377	.04438	1.94	1.18	4.76	2.82	1.69	.7	60.8
64.5	9.98	388	.0443	1.99	1.25	4.75	2.75	1.72	.8	62.8
72.5	9.99	396	.04425	2.03	1.31	4.72	2.69	1.75	.9	64.5
80.5	10.00	403	.04420	2.06	1.35	4.71	2.65	1.78	1.00	65.5
88.5	10.01	409	.04415	2.09	1.40	4.69	2.60	1.80	1.10	67.0
96.5	10.02	414	04411	2.11	1.45	4.66	2.55	1.83	1.20	68.7
105	10.03	420	.04407	2.14	1.50	4.64	2.50	1.86	1.30	70.1
113	10.045	425	.04404	2.16	1.54	4.62	2.46	1.88	1.40	71.3
121	10.05	430	.04400	2.18	1.56	4.62	2.44	1.89	1.50	71.6
129	10.06	435	.04398	2.20	1.61	4.59	2.39	1.92	1.60	73.2
137	10.07	439	.04396	2.22	1.64	4.58	2.36	1.94	1.70	73.9
145	10.085	443	.04393	2.23	1.68	4.55	2.32	1.96	1.80	75.3
153	10.095	447	.04390	2.25	1.70	4.55	2.30	1.98	1.90	75.5
161	10.10	450	.04388	2.26	1.73	4.53	2.27	2.00	2.00	76.5
177.5	10.125	457	.04385	2.29	1.78	4.51	2.22	2.03	2.2	77.7
193.5	10.14	462	.04382	2.31	1.82	4.49	2.18	2.06	2.4	78.7
210	10.165	469	.04378	2.34	1.88	4.46	2.12	2.10	2.6	80.3
226	10.19	474	.04375	2.35	1.90	4.45	2.10	2.12	2.8	80.9
242	10.205	480	.04371	2.38	1.93	4.45	2.07	2.15	3.0	81.1
282	10.26	490	.04366	2.41	2.03	4.38	1.97	2.22	3.5	84.2

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Calcium

Depth ____ Sample 6

Area Correction Factor 1.157

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$
322.5	10.31	501	.04360	2.45	2.10	4.35	1.90	2.29	4.0	85.7
363	10.365	509	.04355	2.47	2.16	4.31	1.84	2.34	4.5	87.4
403	10.42	516	.04351	2.49	2.21	4.28	1.79	2.39	5.0	88.7
443.5	10.475	522	.04349	2.51	2.24	4.27	1.76	2.43	5.5	89.2
483.5	10.53	528	.04345	2.52	2.27	4.25	1.73	2.46	6.0	90.0
524	10.59	535	.04341	2.54	2.29	4.25	1.71	2.49	6.5	90.2
564	10.645	541	.04340	2.55	2.32	4.23	1.68	2.52	7.0	91.3
604.5	10.70	545	.04338	2.56	2.36	4.20	1.64	2.56	7.5	92.2
645	10.76	550	.04335	2.56	2.37	4.19	1.63	2.57	8.0	92.6
685	10.82	554	.04334	2.57	2.38	4.19	1.62	2.59	8.5	92.6
725.5	10.88	558	.04332	2.57	2.39	4.18	1.61	2.60	9.0	93.0
766	10.94	562	.04330	2.57	2.40	4.17	1.60	2.61 ~~7.5~~	9.5	93.4
806	11.00	566	.04328	2.58	2.41	4.17	1.59	2.62	10.0	93.4
846.5	11.06	570	.04327	2.58	2.42	4.16	1.58	2.63	10.5	93.8
886.5	11.13	573	.04325	2.58	2.45	4.13	1.55	2.66	11.0	95.0
927	11.19	576	.04323	2.57	2.45	4.12	1.55	2.66	11.5	95.3
967.5	11.25	580	.04321	2.58	2.47	4.11	1.53	2.68	12.0	95.7
987.5	11.285	581	.04321	2.57	2.48	4.09	1.52	2.69	12.25	96.5
1007	11.31 ~~131~~	582	.04321	2.57	2.48	4.09	1.52	2.69	12.50	96.5

e Content – Initial 56.84

Final 39.67

of Saturation – Initial 98.3

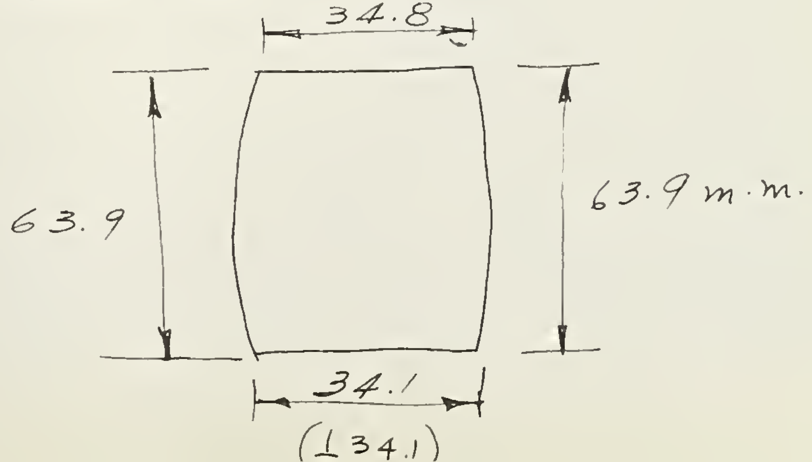
Final 98.0

tio – Initial 1.578

Final 1.105

essure Reaction 87%

Sketch of Failure (⊥34.5)



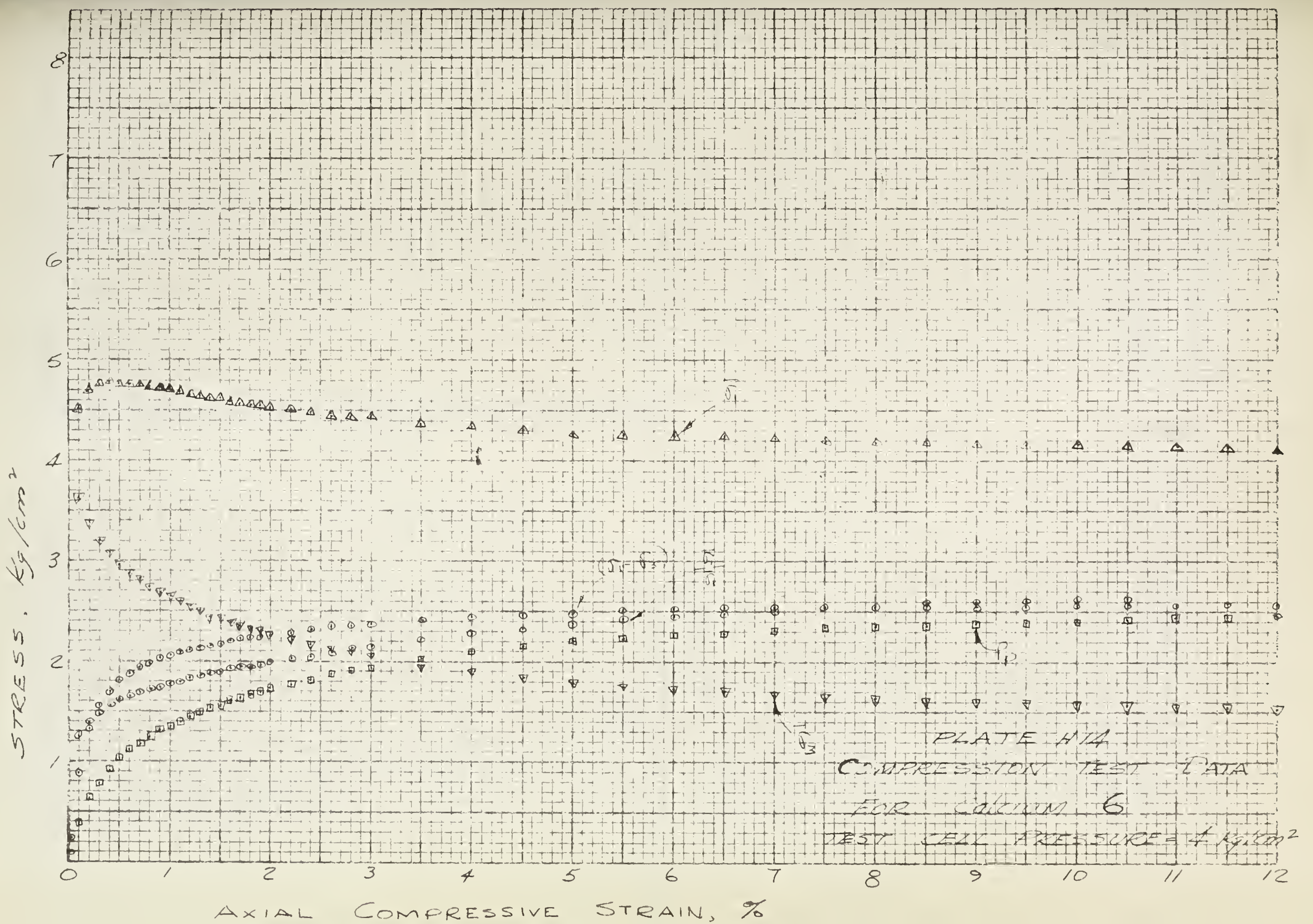


PLATE H14
COMPRESSION TEST DATA
FOR Column 6
TEST CELL PRESSURE = 4 kg/cm²

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Department of Civil Engineering
Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

nple Description CEC = 32.8 me/100 gm Soil
a 95.7, Na 0.7, K 2.5, Mg 1.2
.C. 9.8

ial Weight 134.266 gms

Project THESIS
Hole No. Calcium Gp 2
Depth sample 7
Engineer ST
Technician DW
Date of Set-up 31 Oct 61
Test Lateral Pressure 7.5 kg/cm² (106.5 psi)

h, mm 1. 80.44 2. 80.44 Aver: 80.44
mm Top 1. 35.34 2. 35.32 Aver: 35.33 Area Top A_T 9.803 sq.mm
Centre 1. 35.60 2. 35.64 Aver: 35.62 Area Centre A_C 9.965 sq.mm
Bottom 1. 35.92 2. 35.86 Aver: 35.89 Area Bottom A_B 10.117 sq.mm
al Volume 80.12 cc

Average X-Sect Area $= \frac{1}{4}(A_T + 2A_C + A_B)$ = 9.96 cm²

OLIDATION DATA

	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
1330	0	25.00	
	.25	24.70	
	.5	24.45	
	1	23.90	
	1.5	23.45	
	2	23.05	
	3	~~23~~ 22.35	
	4	21.70	
	5	21.15	
	6	20.60	
	8	19.70	
	10	18.90	
	15	17.80	
	22	16.10	
	25	15.50	
	30	14.65	
	35	13.95	
	40	13.35	
	45	12.85	
	51	12.30	

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
1430	60	11.60	
	100	9.7	
1600	150	8.50	
1650	200	7.90	
1910	340	7.29	
2200	510	7.05	
1 Nov 0830	1140	6.80	
2 Nov 0915	2625	6.60	
3 Nov 0900	4050	6.50	
4 Nov 1330	5760	6.40	

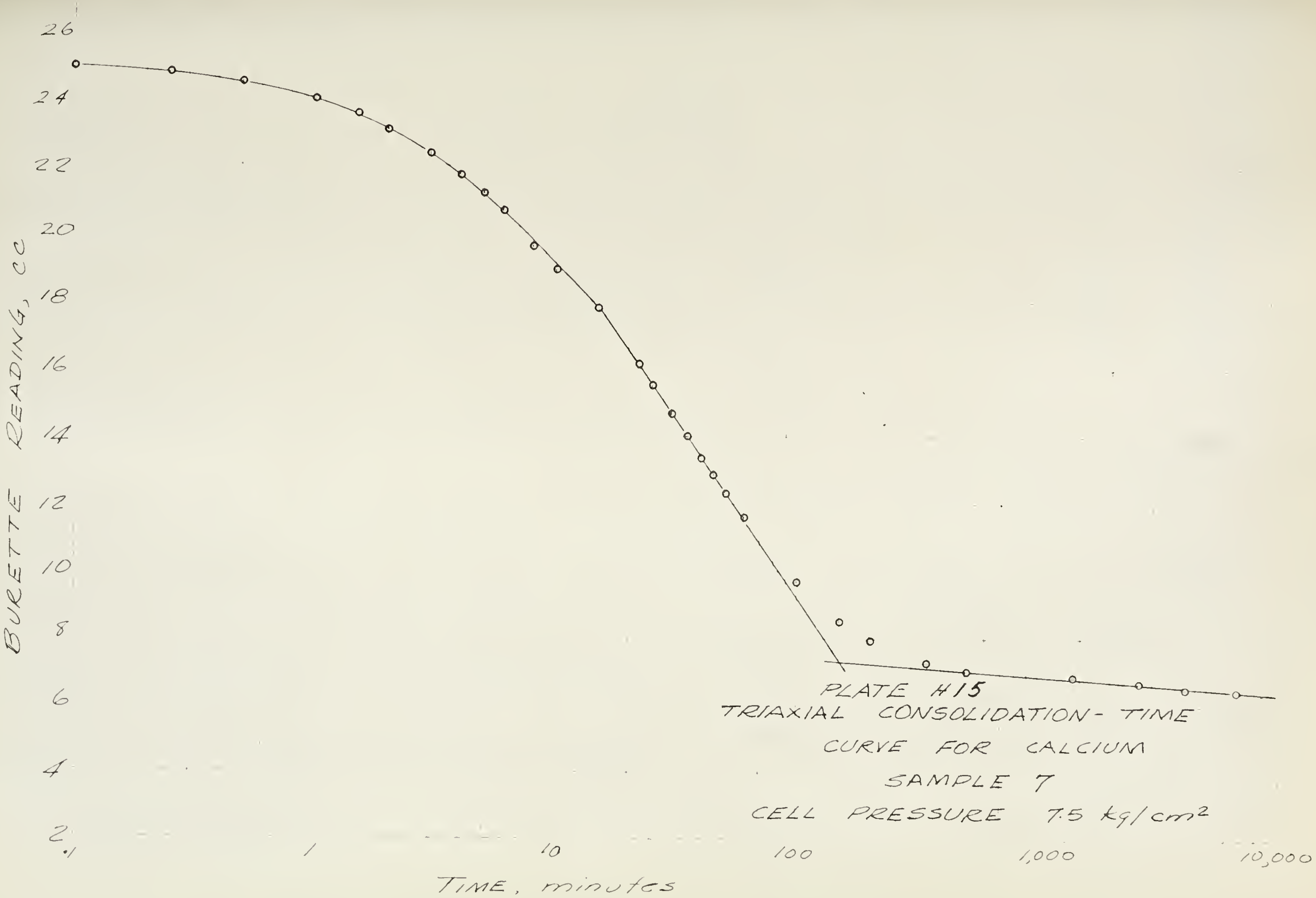

PLATE H15
TRIAXIAL CONSOLIDATION - TIME
CURVE FOR CALCIUM
SAMPLE 7
CELL PRESSURE 7.5 kg/cm^2

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Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. Calcium
Depth Sample 7
Engineer ST
Technician –
Date of Test 16 Nov 61
Test Lateral Pressure σ_3 7.5 kg/cm²
Back Pressure 2.0
Remarks Rate of Strain
300.4 min/10 m.m.
Proving Ring 3-1498
Area Correction Factor K = 1.196

Failure
σ_3) = 4.98 σ_1 = 12.48
3.84 kg/cm²
8.64
3.66
8.5 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}}{P_p}{\sigma_1 - \sigma_3}$
	9.96		0	0	−.05					
2	9.96	40	.0511	.245	−.02					
4	9.965	60	.0501	.361	0.0					
8	9.97	137	.0475	.78	.12	8.16	7.38	1.106	.1	15.4
16	9.98	307	.04497	1.65	.45	8.70	7.05	1.234	.2	27.3
24	9.99	450	.04388	2.36	.80	9.06	6.70	1.352	.3	33.9
32	10.00	553	.04334	2.86	1.10	9.26	6.40	1.447	.4	38.5
40	10.01	628	.04303	3.23	1.34	9.39	6.16	1.524	.5	41.5
48	10.02	684	.04282	3.50	1.54	9.46	5.96	1.587	.6	44.0
56	10.03	720	.04268	3.66	1.70	9.46	5.80	1.631	.7	46.4
64	10.04	753	.04257	3.82	1.82	9.50	5.68	1.673	.8	47.6
72	10.05	780	.04248	3.94	1.93	9.51	5.57	1.707	.9	48.9
80.5	10.06	803	.04240	4.05	2.07	9.48	5.43	1.746	1.0	51.1
96.5	10.08	838	.04226	4.20	2.21	9.49	5.29	1.794	1.2	52.6
112.5	10.10	864	.04218	4.32	2.37	9.45	5.13	1.842	1.4	54.9
129	10.12	886	.04210	4.41	2.50	9.41	5.00	1.882	1.6	56.7
145	10.14	903	.04204	4.48	2.60	9.38	4.90	1.914	1.8	58.0
161	10.16	919	.04199	4.54	2.68	9.36	4.82	1.942	2.0	59.0
181	10.19	937	.04192	4.61	2.80	9.31	4.70	1.981	2.25	60.7
201	10.215	950	.04189	4.66	2.91	9.25	4.59	2.015	2.50	62.4
241	10.27	973	.04180	4.74	3.06	9.18	4.44	2.068	3.0	64.5
281.5	10.32	995	.04173	4.81	3.21	9.10	4.29	2.121	3.5	66.7
322	10.375	1011	.04170	4.86	3.30	9.06	4.20	2.157	4.0	67.9
362	10.43	1026	.04164	4.90	3.40	9.00	4.10	2.195	4.5	69.4
410	10.485	1041	.04160	4.94	3.54	8.90	3.96	2.247	5.1	71.7
442.5	10.54	1050	.04157	4.95	3.56	8.89	3.94	2.256	5.5	71.9
482.5	10.60	1060	.04154	4.97	3.60	8.87	3.90	2.274	6.0	72.4
523	10.65	1068	.04151	4.98	3.67	8.81	3.83	2.300	6.5	73.7
563	10.71	1077	.04150	4.99	3.72	8.77	3.78	2.320	7.0	74.5

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Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. Calcium

Depth Sample 7

Area Correction Factor 1.196

	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma 1 - \sigma 3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm2 P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\, P_F}{\sigma_1 - \sigma_3}$
3	603	10.77	1084	.04148	4.99	3.75	8.74	3.75	2.331	7.5	75.1
6	643.5	10.83	1090	.04146	4.99	3.80	8.69	3.70	~~8~~ 2.349	8.0	76.2
4	700	10.885	1095	.04144	4.98	3.84	8.64	3.66	2.361	8.5	77.1
3	724	10.945	1095	.04144	4.96	3.83	8.63	3.67	2.351	9.0	77.2
5	764	11.005	1094	.04144	4.93	3.84	8.59	3.66	2.347	9.5	77.9
8	804.5	11.07	1089	.04146	4.88	3.86	8.52	3.64	2.341	10.0	79.1
0	844.5	11.13	1081	.04149	4.82	3.87	8.45	3.63	2.328	10.5	80.3
3	885	11.19	1067	.04151	4.73	3.87	8.36	3.63	2.303	11.0	81.8

Content – Initial 52.2

Final 30.89

of Saturation – Initial 97.4

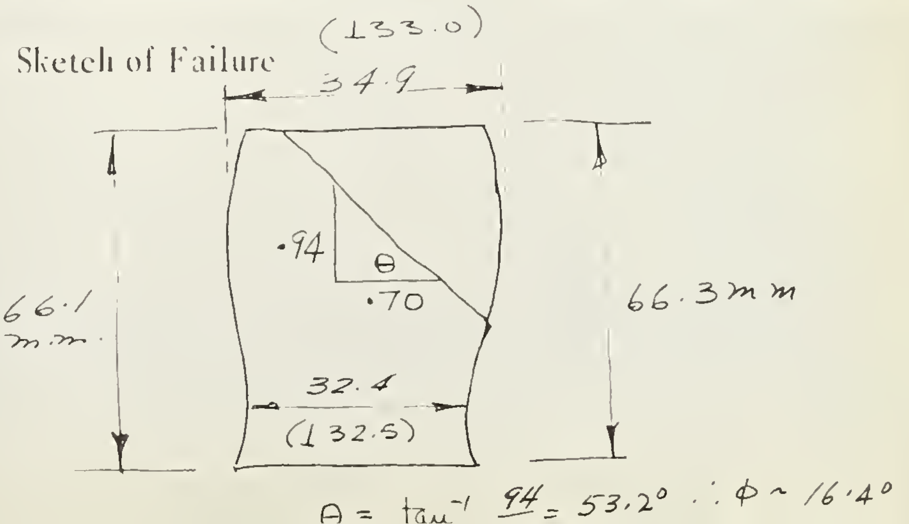
Final 98.3

io – Initial 1.501

Final 0.858

ssure Reaction 80%

Sketch of Failure



$\theta = \tan^{-1} \frac{94}{70} = 53.2^\circ \therefore \phi \sim 16.4^\circ$

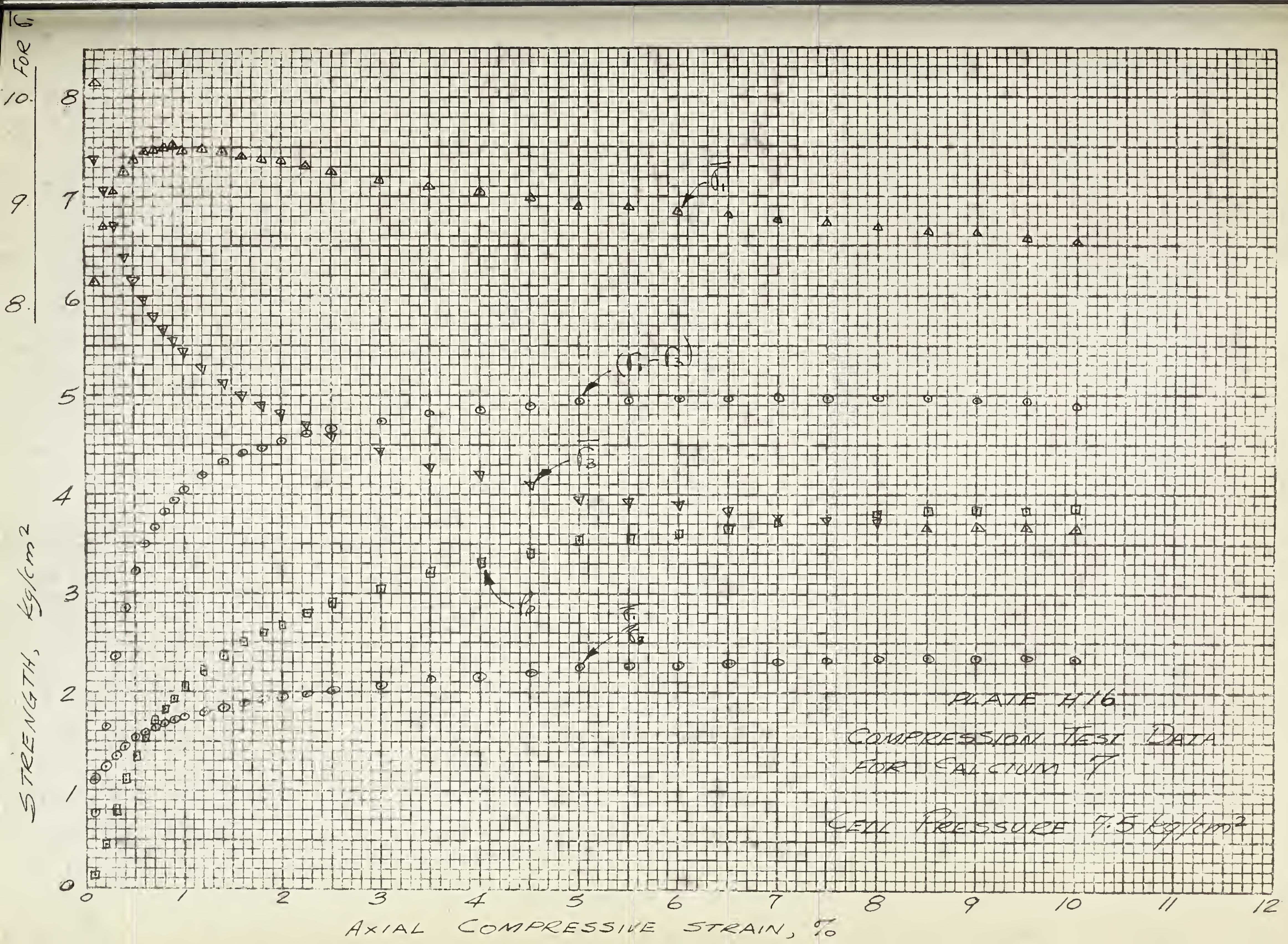


PLATE H16

COMPRESSION TEST DATA
FOR CALCIUM 7

CELL PRESSURE 7.5 kg/cm²

APPENDIX I

DATA SHEETS FOR TESTS ON SODIUM MODIFIED SOIL

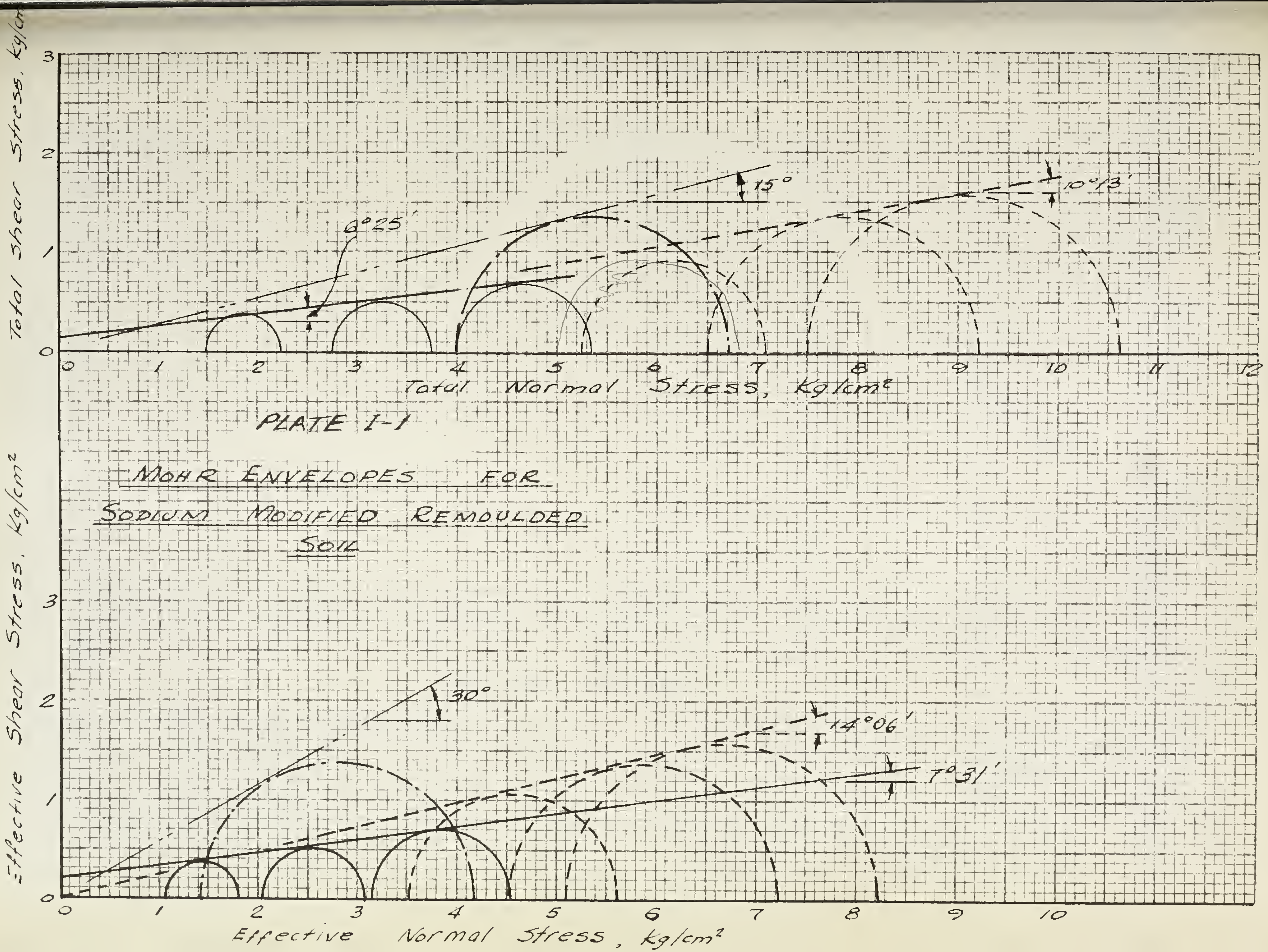


PLATE I-1

MOHR ENVELOPES FOR SODIUM MODIFIED REMOULDED SOIL

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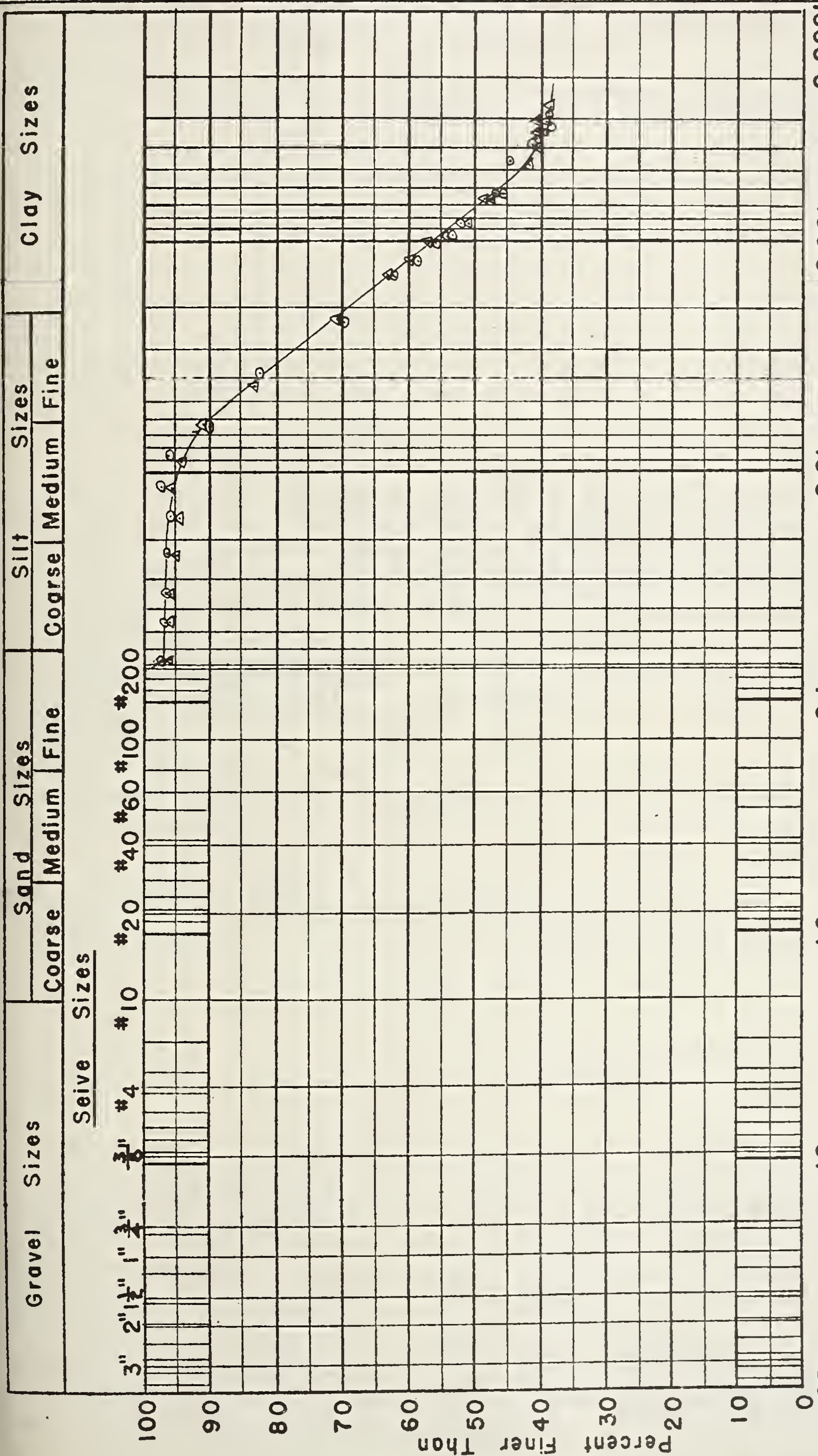
SOIL MECHANICS LABORATORY

GRAIN SIZE CURVE

PROJECT THESIS	
SITE	
SAMPLE SODIUM (Gpl)	
LOCATION	
HOLE	DEPTH
TECHNICIAN ST/DW	DATE 18 July 61



D₁₀ = ____ mm.
D₆₀ = ____ mm.
Cu ____

Legend
⊙ Sample #1
△ Sample #2

Remarks: FROM WORK OF HAMILTON
WL = 95.7
Wp = 26.4
Ip - 69.3
Compression Index = 0.98

Note: M·I·T· Grain Size Scale

PLATE I-2

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SOIL MECHANICS LABORATORY

HYDROMETER TEST

PROJECT THESIS
SITE
SAMPLE SODIUM #2 (Gp2)
LOCATION
HOLE DEPTH
TECHNICIAN DW DATE 28 June 61

Date	Temp. °C	Time	Elapsed Time	R'_h	$R_h = R'_h + c_m$	D m.m.	$R_h+m_t-c_d$	W.%	W % Basis Orig. Sample	Remarks
28 JUN		1407	.5	18.0	18.5	.067	18.9	95.7		
			1 min	18.0	18.5	.048	18.9	95.7		
			2	17.9	18.4	.035	18.8	95.1		
			4	17.8	18.3	.024	18.7	94.6		
	23.7		8	17.7	18.2	.017	18.6	94.1		
			15	18.0	18.5	.0125	18.9	95.7		
	23.6		30	17.7	18.2	.009	18.6	94.1		
	23.4		60	17.1	17.6	.0063	18.0	91.1		
	23.0		2h:20m	15.7	16.2	.0043	16.5	83.5		
28 JUN	22.4	2207	8:00	13.2	13.7	.0023	13.9	70.3		
29 ✓	21.8	.0842	18:35	11.9	12.4	.00155	12.4	62.8		
29 ✓	22.3	1632	26:25	11.1	11.6	.0013	11.7	59.2		
30 ✓	21.2	0837	42.30	10.7	11.2	.00108	11.1	56.2		
30 ✓	21.2	1642	50.35	10.2	10.7	.00098	10.6	53.6		
1 July	20.5	1127	69:20	9.9 8.0	10.4 8.5	00085 00050	10.2 8.3	51.6 42.0		
2 July	20.0	2200	103.53	9.5 8.0	10.0 8.5	00069 00040	9.7 8.2	49.1 41.5		
3 ✓	20.1	1015	116:08	9.2 7.9	9.7 8.4	00066 00039	9.4 8.1	47.6 41.0		
3 ✓	20.4	2120	127:13	9.0 7.7	9.5 8.2	00063 00038	9.9 7.9	47.1 40.5		
4 ✓	21.4	1107	141:00	7.5	8.0	.00035	8.0	40.5		
5 ✓	22.4	0832	162:25	7.3	7.8	.00031	8.0	40.5		
6 July	23.8	0842	186:35	7.0	7.5	.00030	7.7	39.0		

Hydrometer No.s 9857R and 50700R Graduate No.

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s-1} (R_h + m_t - c_d) =$ ~~0.5~~ $(R_h + m_t - c_d)$

Meniscus correction = c_m = 0.5 and 0.5 respectively

Dispersing agent used Calgon Amount 10 ml

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = 0.3 and 0.3 respectively

Specific Gravity of Solids = G_s 2.79

Description of Sample

Method of Preparation

Remarks

Initial Moisture Content

Container No.
Wt. Sample Wet + Tare
Wt. Sample Dry + Tare
Wt. Water
Tare Container
Wt. of Dry Soil
Initial Moisture w %

Dry Weight of Sample

Container No.
Wt. Sample (Wet/Dry) + Tare
Tare
Wt. (Wet/Dry) Soil
Dry Weight from Initial Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist. \%}}$ =

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HYDROMETER TEST

PROJECT THESIS
SITE
SAMPLE SODIUM #1 (Gp1)
LOCATION
HOLE DEPTH
TECHNICIAN DW DATE 28 June '61

Date	Temp· °C	Time	Elapsed Time	R_h'	$R_h = R_h' + c_m$	D m.m.	$R_h + m_t - c_d$	W·%	W % Basis Orig. Sample	Remarks
28 JUN		1350	·5 m	20.0	20·5	0.067	20.9	97.0		
			1 min	19.9	20.4	·0 47	20·8	96·5		
			2	19·8	20.3	·034	20.7	96·0		
			4	19·8	20.3	·02 4	20·7	96·0		
	23·6		8	19·7	20·2	,017	20·6	95·6		
			15	20.0	20·5	·012	20·9	97·0		
	23·6		30	19·8	20·3	·0087	20·7	96·0		
	23·3		60	18·6	19·1	·0062	19·4	90·0		
	23·0		2h:34m	17·0	17·5	·0039	17·8	82·6		
	22·4	2150	8:00	14·4	14·9	·0023	15·1	70·0		
29 JUN	21·9	0840	18:50	12·8	13·3	·0015	13·4	62·2		
✓	22·2	1630	26:40	12·1	12·6	·0013	12·7	58·9		
30 ✓	21·2	·0835	42.45	11·6	12·1	·00106	12·0	55·7		
30 ✓	21·2	16 40	50·50	11·2	11·7	·00096	11·6	53·8		
1 July '61	20·3	1130	69:40	11·0 9·4	11·5 9·9	00083 00047	11·3 9·7	52·4 45·0		
2 July	20·6	2200	104:50	10·3 8·8	10·8 9·3	00069 00038	10·5 9·0	48·7 41·8		
3 ✓	20·6	1015	116:25	10·0 8·5	10·5 9·0	00065 00037	10·3 8·8	47·8 40·8		
3 ✓	20·6	2115	127:25	9·8 8·3	10·3 8·8	00063 00035	10·1 8·6	46·7 39·9		
4 ✓	21 4	1105	141:15	8·0	8·5	·00034	8·5	39·4		
5 ✓	22·4	0830	162.40	7·9	8·4	·00031	8·6	39·9		

Hydrometer No.s 9857R and 50700R Graduate No. ______

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s - 1}(R_h + m_t - c_d) = $ ______ $(R_h + m_t - c_d)$

Meniscus correction = c_m = 0·5 and 0·5 respectively

Dispersing agent used calgon Amount 10 ml

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = 0·3 and 0·3 respectively

Specific Gravity of Solids = G_s 2.79

Description of Sample ______

Method of Preparation ______

Remarks ______

Initial Moisture Content

Container No. ______
Wt. Sample Wet + Tare ______
Wt. Sample Dry + Tare ______
Wt. Water ______
Tare Container ______
Wt. of Dry Soil ______
Initial Moisture w % ______

Dry Weight of Sample

Container No. ______
Wt. Sample (Wet/Dry) + Tare 1046·3
Tare 1012·7
Wt. (~~Wet~~/Dry) Soil 33·6
Dry Weight from Initial Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist.}\%}$ = ______

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SPECIFIC GRAVITY

PROJECT THESIS
SITE
SAMPLE SODIUM Gp 1
LOCATION
HOLE DEPTH
TECHNICIAN DW + ST DATE 14 July 61

Sample No.	1	2
Flask No.	100	101
Method of Air Removal	Vacuum	Vacuum
W_{b+w+s}	693.63	691.83
Temperature T	28·1 °C	28·8 °C
W_{b+w}	669.03	666.82
Evaporating Dish No. (beaker)	100	101
Wt. Sample Dry + Dish	223·12	216·03
Tare Dish	184.83	177.07
W_s	38·29	38·96
G_s	2.80	2.79

W_{b+w+s} = Weight of flask + water + sample at T°.
W_{b+w} = Weight of flask + water at T° (flask calibration curve).
W_s = Weight of dry soil
G_s = Specific gravity of soil particles $= \dfrac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.			Sample No.		
Container No.			Container No.		
Wt. Sample Wet + Tare			Wt. Test Sample Wet + Tare		
Wt. Sample Dry + Tare			Tare Container		
Wt. Water			Wt. Test Sample Wet		
Tare Container			W_s		
Wt· of Dry Soil					
Moisture Content w %					

Description of Sample:

Remarks:

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Department of Civil Engineering
Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

mple Description Cation Exchange Capacity 2.9 me/100 gm soil, Average Adsorbed ations: Ca 5.3%, Na 87.9%, K 3.4, 3.4, Salt Content of Pore ater 0

tial Weight 124.224 gms

Project THESIS
Hole No. SODIUM (GP1)
Depth sample 1
Engineer S.T.
Technician D.W.
Date of Set-up 26 JUN 61
Test Lateral Pressure 1.5 kg/cm²

	1.	2.	Aver:		
th, mm	80.48	80.48	80.48	Area Top A_T	~~8~~9.682 sq.mm
. mm Top	35.10	35.12	35.11	Area Centre A_C	9.898 sq.mm
Centre	35.60	35.40	35.50	Area Bottom A_B	10.066 sq.mm
Bottom	35.84	35.76		Average X-Sect Area	

nal Volume 79.56 cc

Average X-Sect Area $= \frac{1}{4}(A_T + 2A_C + A_B)$ = 9.89

SOLIDATION DATA

e e	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
✓0900	0	24.10	
	1	23.90	
	2 ~~8~~	23.87	
	4	23.85	
	6	23.82	
	8	23.80	
	10	23.80	
	20	23.73	
	40	23.65	
	60	23.60	
	100	23.45	
	150	23.35	
	255	23.15	
1540	400	22.98	
1715	495	22.85	
✓0835	1415	22.25	
2210	2230	21.90	
0835	2805	21.~~8~~70	
0845	4305	21.30	
11 ~~0845~~	5895	21.00	

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
2 Jul 2215	7995	20.68	
3 ✓ 0835	8615	20.57	
4 ✓ 0840	10,060	20.30	
5 ✓ 0910	11,530	20.15	
6 ✓ 0830	12,930	19.90	
7 ✓ 0830	14,370	19.75	
8 ✓ 0900	15,840	19.60	
10 ✓ ✓	18,720	19.35	
11 ✓ 0840	20,140	19.25	
12 ✓ 0840	21,580	19.15	
13 ✓ 0840	23,020	19.02	
14 ✓ ✓	24,460	18.94	
15 ✓ 2215	26,715	18.80	
17 ✓ 0810	28,750	18.71	
18 ✓ 0830	30,210	18.67	
20 ✓ 0850	33,110	18.52	
22 ✓ 0945	36,045	18.40	
24 ✓ 0810	38,830	18.30	
25 ✓ 1230	40,530	18.30	
26 ✓ 1045	41,865	18.26	

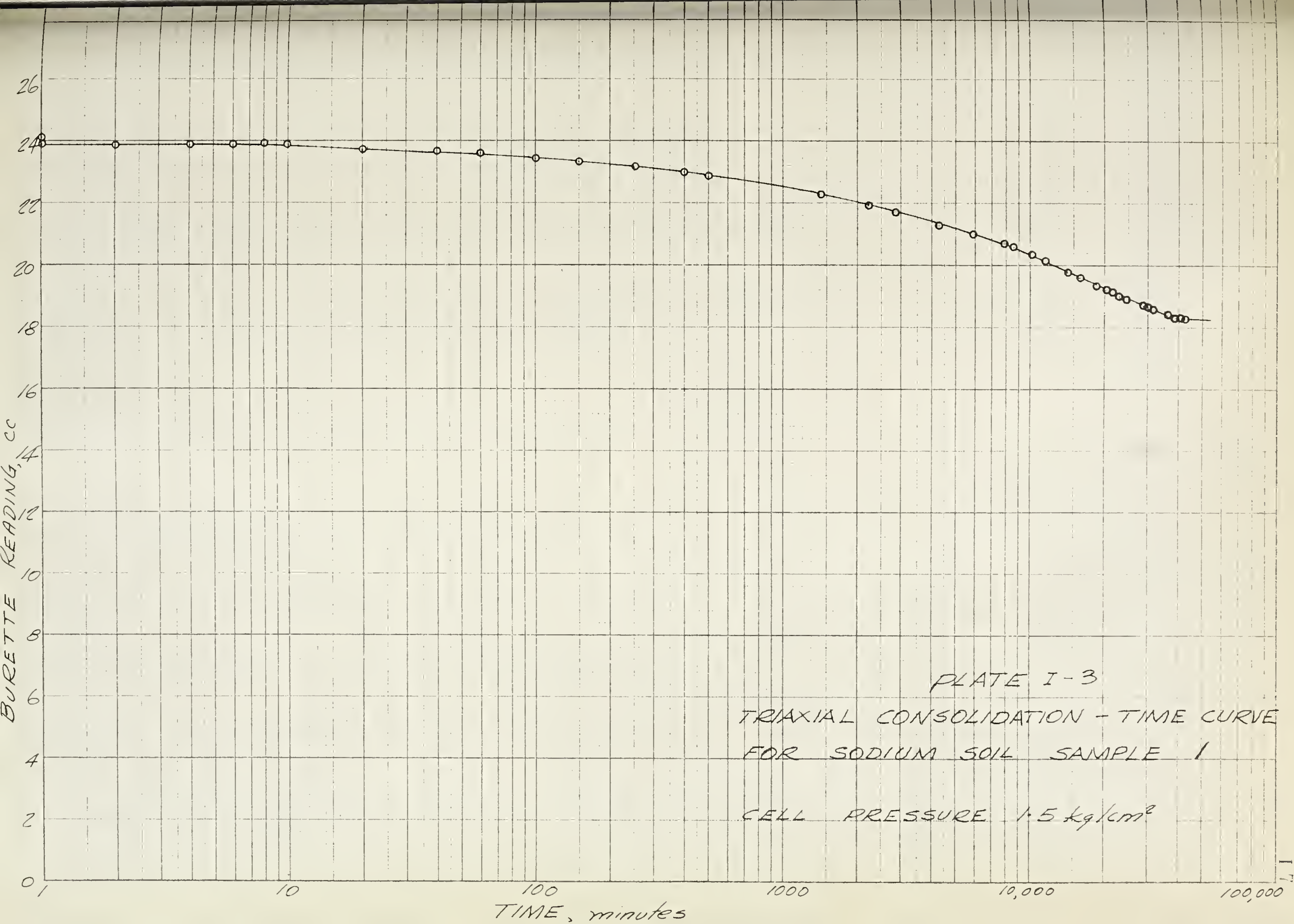

PLATE I-3

TRIAXIAL CONSOLIDATION - TIME CURVE
FOR SODIUM SOIL SAMPLE 1

CELL PRESSURE 1.5 kg/cm²

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Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. SODIUM (Gp 1)
Depth Sample 1
Engineer ST
Technician DW
Date of Test 27 July 61
Test Lateral Pressure σ3 1.5 Kg/cm²
Back Pressure 2.0 "
Remarks Proving Ring 3-1498 (50 kg)
Rate of Strain 408 min/10 mm (≈ 1.35%/hr)
Area Correction Factor K = 1.05

Failure
σ3) = 0.74 σ1 = 2.24
0.44 Kg/cm²
1.80 ✓
1.06
3.2 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_p}{\sigma_1 - \sigma_3}$
0	9.89	0		0	−.03					
9	9.89	20	.0525	.11	0	1.61	1.50	1.07	.11	
16	9.90	40	.0511	.22	+.07	1.65	1.43	1.15	.20	31.8
27	9.92	60	.0505	.32	+.15	1.67	1.35	1.24	.34	46.8
40	9.94	80	.0493	.42	.19	1.73	1.31	1.32	.50	45.2
48	9.95	90	.04895	.46	.24	1.72	1.26	1.37	.6	52.2
56	9.96	100	.0486	.51	.25	1.76	1.25	1.41	.7	49.0
64	9.97	106	.0484	.54	.26	1.78	1.24	1.44	.8	48.1
72	9.98	113	.0482	.57	.31	1.76	1.19	1.48	.9	54.4
80	9.99	118	.0480	.60	.33	1.77	1.17	1.51	1.0	55.0
88	10.00	122	.0479	.61	.33	1.78	1.17	1.52	1.1	54.1
96	10.01	126	.0478	.63	.34	1.79	1.16	1.54	1.20	53.9
104	10.02	129	.04773	.65	.34	1.81	1.16	1.56	1.30	52.3
112	10.03	132	.04767	.66	.36	1.80	1.14	1.58	1.40	54.5
121	10.04	136	.04754	.68	.38	1.80	1.12	1.61	1.50	55.8
129	10.05	139	.04745	.69	.39	1.80	1.11	1.62	1.60	56.5
137	10.06	141	.04740	.70	.40	1.80	1.10	1.64	1.70	57.1
145	10.07	143	.04738	.71	.40	1.81	1.10	1.65	1.80	56.3
153	10.08	145	.04732	.71	.40	1.81	1.10	1.65	1.90	56.3
1.61	10.09	147	.04728	.72	.42	1.80	1.08	1.67	2.0	58.3
1.69	10.10	148	.04726	.73	.42	1.81	1.08	1.68	2.1	57.5
177	10.11	149	.04724	.73	.42	1.81	1.08	1.68	2.2	57.5
185	10.12	150	.04721	.73	.42	1.81	1.08	1.68	2.3	57.5
193	10.13	150	✓	.73	.42	1.81	1.08	1.68	2.4	57.5
209	10.155	152	.04718	.74	.43	1.81	1.07	1.69	2.6	58.1
225.5	10.18	152	.04718	.74	.43	1.81	1.07	1.69	2.8	58.1
241.5	10.19	152	.04718	.74	.43	1.81	1.07	1.69	3.0	58.1
257.5	10.21	152	.04718	.74	.44	1.80	1.06	1.70	3.2	59.4
273.5	10.24	151	.04721	.73	.44	1.79	1.06	1.69	3.4	60.3

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Department of Civil Engineering
Soil Mechanics Laboratory
XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. SODIUM Gp 2
Depth ______ Sample 1
Area Correction Factor 1.05

	Strain Dial Div.	A_c cm2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm2 P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$
3	273.5	10.24	151	.04721	.73	.44	1.79	1.06	1.69	3.4	60.3
	290	10.26	149	.04724	.72	.44	1.78	1.06	1.68	3.6	61.1
8	306	10.28	147	.04728	.71	.45	1.76	1.05	1.68	3.8	63.4
5	322	10.30	145	.04732	.70	.45	1.75	1.05	1.67	4.0	64.3
	342	10.33	143	.04738	.69	.45	1.74	1.05	1.66	4.25	65.2
2	362	10.355	138	.04750	.66	.45	1.71	1.05	1.63	4.50	68.2
	382	10.38	131	.04768	.63	.44	1.69	1.06	1.59	4.75	69.8
7	402.5	10.41	124	.04788	.60	.42	1.68	1.08	1.56	5.0	70.0
6	442.5	10.46	114	.04815	.55	.39	1.66	1.11	1.50	5.5	70.9
2	483	10.52	106	.0484	.51	.38	1.63	1.12	1.46	6.0	74.5

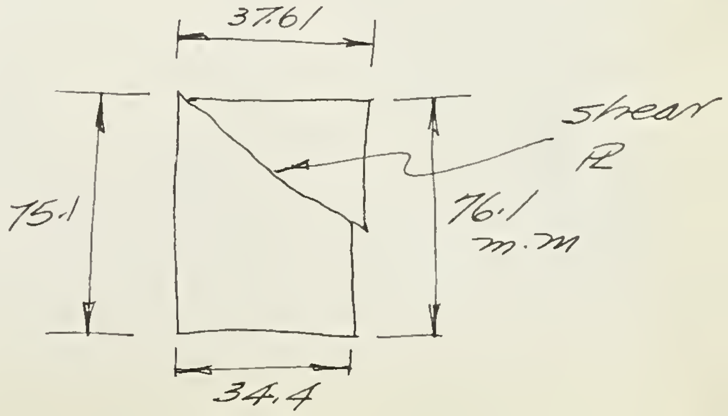
e Content – Initial 77.11
Final 69.67

of Saturation – Initial ~~2.165~~ 99.4%
Final 100 (Assumed)

atio – Initial 2.165
Final 1.94 (based on e=W·Gs)

essure Reaction 96%

Sketch of Failure



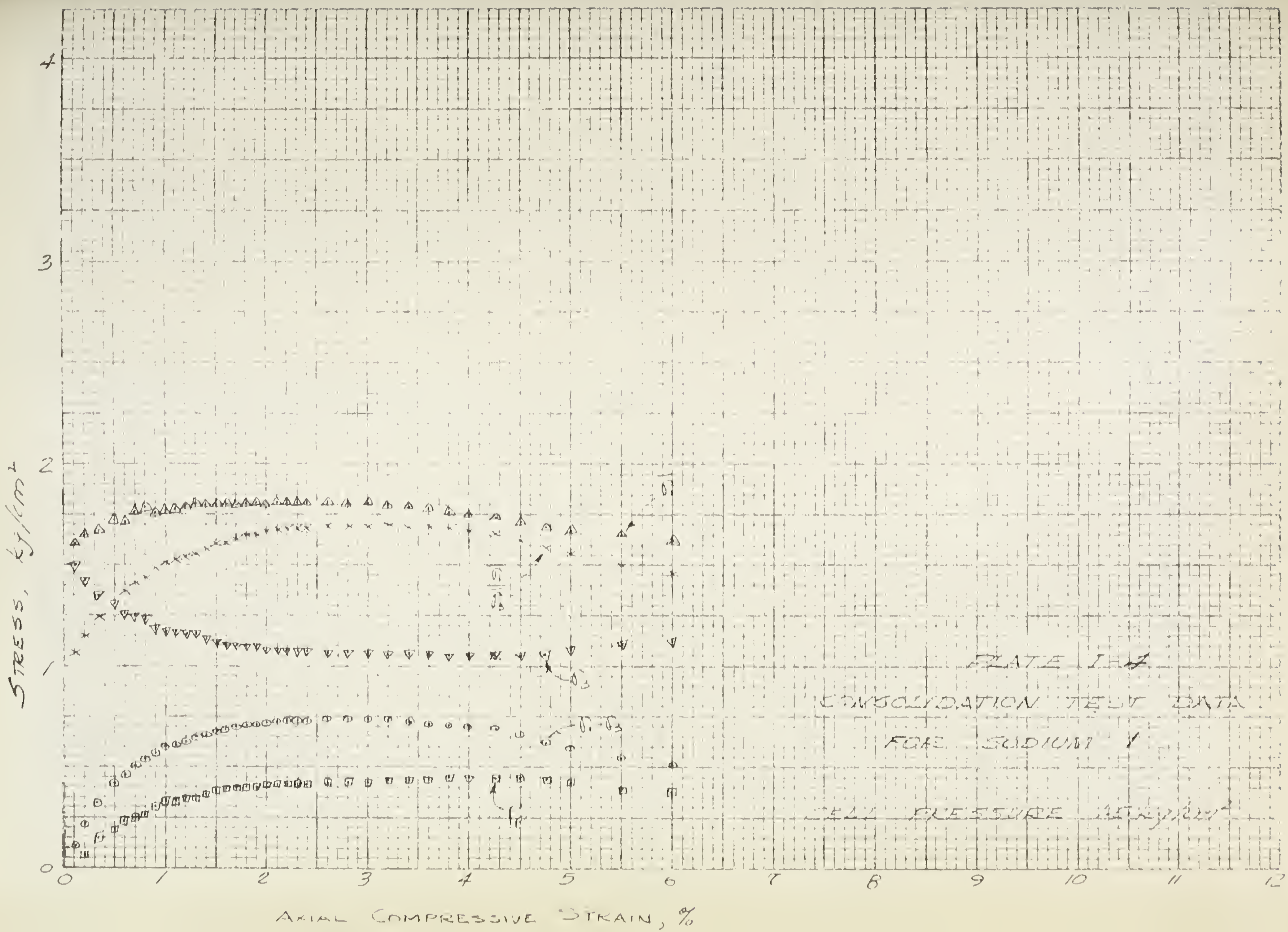


PLATE I 4
CONSOLIDATION TEST DATA
FOR SODIUM 1
CELL PRESSURE 1.5 kg/cm²
STRESS, kg/cm²
AXIAL COMPRESSIVE STRAIN, %
0
1
2
3
4
5
6
7
8
9
10
11
12
σ1
σ1/σ3
σ3
σ1-σ3
p

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. SODIUM Gp I
Depth sample 2
Engineer ST
Technician DW
Date of Set-up 26 JUN 61
Test Lateral Pressure 2.75 kg/cm²

mple Description CEC 32.9 me/100 gm soil
ver Adsorbed Cations : Ca 5.3%,
/a 87.9%, K 3.4%, Mg 3.4%
5.C = 0

tial Weight 115.247 gms

th, mm 1. 72.70 2. 72.72 Aver: 72.71 Area Top A_T 10.179 sq.mm
. mm Top 1. 36.00 2. 36.00 Aver: 36.00 Area Centre A_C 9.931 sq.mm
Centre 1. 35.52 2. 35.60 Aver: 35.56 Area Bottom A_B 9.621 sq.mm
Bottom 1. 35.00 2. 35.00 Aver: 35.00 Average X-Sect Area
nal Volume 72.10 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$
$=$ 9.92

SOLIDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
1 0900	0	24.70	
	1	24.20	
	2	24.20	
	4	24.10	
	6	24.05	
	8	24.00	
	10	23.95	
	20	23.80	
	40	23.60	
	60	23.45	
	100	23.20	
	150	22.98	
1315	255	22.65	
1540	400	22.35	
0835	1415	21.30	
2210	2230	20.75	
0835	2855	20.45	
45	4305	19.80	
1115	5895	19.25	
835	8615	18.50	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
4 July 0840	10,060	18.15	
5 ✓ 0910	11,530	17.80	
6 ✓ 0830	12,930	17.44	
7 ✓ 0830	14,370	17.10	
8 ✓ 0900	15,840	16.82	
10 ✓ ✓	18,720	16.35	
11 ✓ 0840	20,140	16.15	
12 ✓ 0840	21,580	16.00	
13 ✓ ✓	23,020	15.80	
14 ✓ ✓	24,460	15.60	
15 ✓ 2215	26,715	15.35	
17 ✓ 0810	28,750	15.20	
18 ✓ 0830	30,210	15.05	
19 ✓ 0850	31,670	14.92	
20 ✓ ✓	33,110	14.70	
22 ✓ 0945	36,045	14.55	
24 ✓ 0810	38,830	14.40	
25 ✓ 1230	40,530	14.30	
26 ✓ 1045	41,865	14.25	
27 ✓ 1215	43,395	14.20	

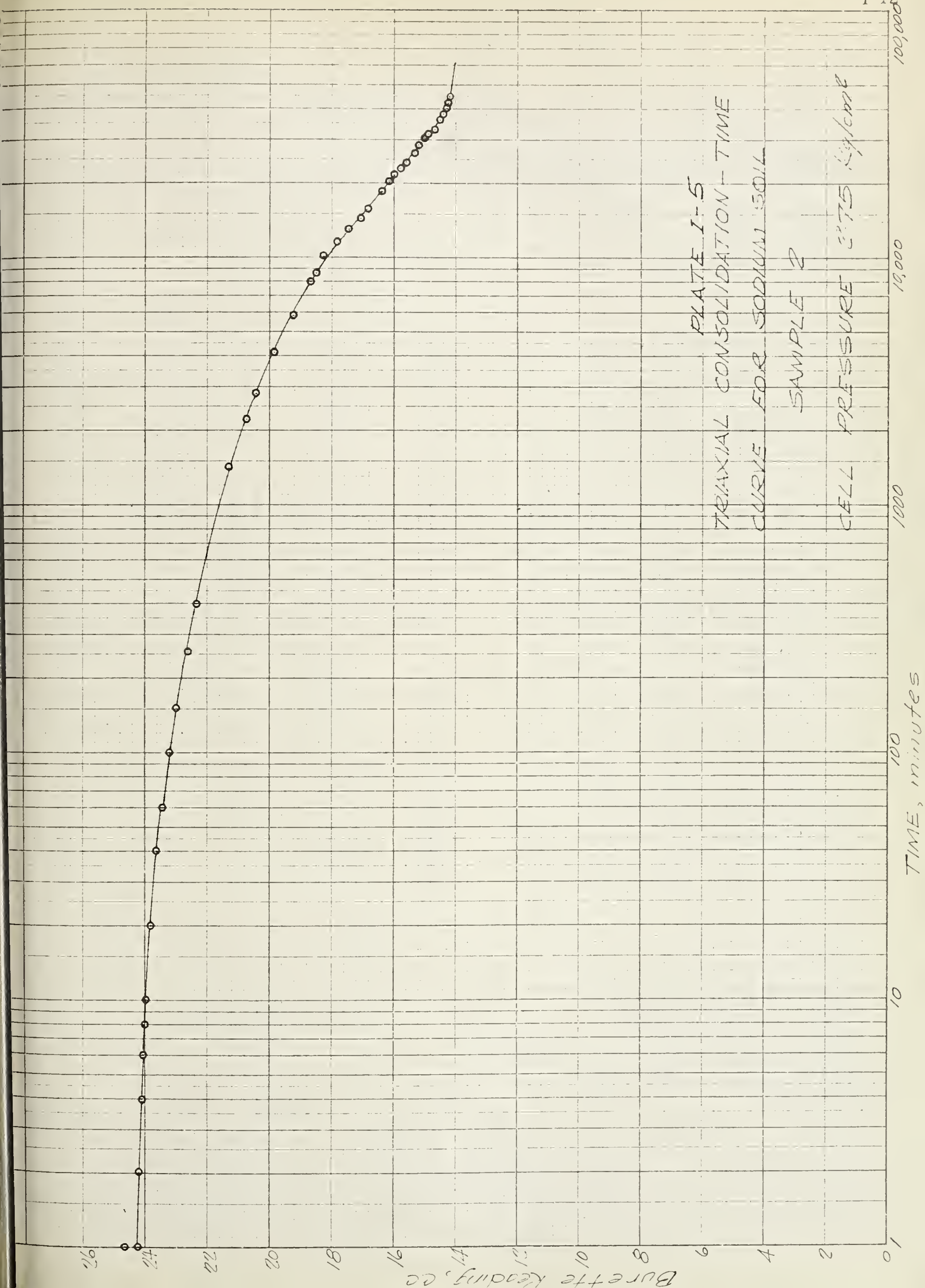


PLATE I-5

TRIAXIAL CONSOLIDATION-TIME CURVE FOR SODIUM SOIL

SAMPLE 2

CELL PRESSURE 3.75 kg/cm^2

UNIVERSITY OF ALBERTA

Department of Civil Engineering
Soil Mechanics Laboratory

AXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. SODIUM Gpl
Depth Sample 2
Area Correction Factor $K = 1.10$
Test Lateral Pressure 2.75 kg/cm²

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_P}{\sigma_1-\sigma_3}$ %
0	9.92			0	+.06					
4	9.92	20	.0525	.12	.16	2.71	2.59	1.05	.055	
8	9.93	40	.0511	.23	.22	2.76	2.53	1.09	.11	
14.5	9.94	75	.0495	.41	.32	2.84	2.43	1.17	.2	78.0
22	9.95	104	.04938	.57	.44	2.88	2.31	1.25	.3	77.1
29	9.96	124	.04788	.66	.49	2.92	2.26	1.29	.4	74.2
36.5	9.965	140	.04744	.73	.52	2.96	2.23	1.33	.5	71.2
43.5	9.975	152	.04717	.79	.54	3.00	2.21	1.36	.6	68.3
51	9.985	163	.04693	.84	.57	3.02	2.18	1.39	.7	67.8
58	9.995	171	.04678	.88	.60	3.03	2.15	1.41	.8	68.2
65.5	10.00	179	.04663	.92	.62	3.05	2.13	1.43	.9	67.4
73	10.01	187	.04650	.96	.64	3.07	2.11	1.45	1.0	66.7
80	10.02	192	.04640	.98	.66	3.07	2.09	1.47	1.1	67.3
94.5	10.04	196	.04635	.99	.69	3.06	2.06	1.49	1.3	69.0
101.5	10.05	197	.04633	1.00	.69	3.06	2.06	1.49	1.4	69.0
109	10.06	197	.04633	1.00	.69	3.06	2.06	1.49★	1.5	69.0
116.5	10.065	196	.04635	.99	.69	3.05	2.06	1.48	1.6	69.7
123.5	10.075	194	.04639	.98	.69	3.04	2.06	1.48	1.7	70.4
131	10.085	192	.04640	.97	.68	3.04	2.07	1.47	1.8	70.1
138	10.095	190	.04644	.96	.66	3.05	2.09	1.46	1.9	68.7
145.5	10.105	189	.04648	.96	.66	3.05	2.09	1.46	2.0	68.7
163.5	10.13	183	.04658	.93	.66	3.02	2.09	1.44	2.25	71.0
182	10.15	178	.04664	.90	.65	3.00	2.10	1.43	2.5	72.2
218	10.195	170	.04662	.86	.64	2.97	2.11	1.41	3.0	74.4
255	10.245	163	.04694	.82	.63	2.94	2.12	1.39	3.5	76.8

★ taken as failure

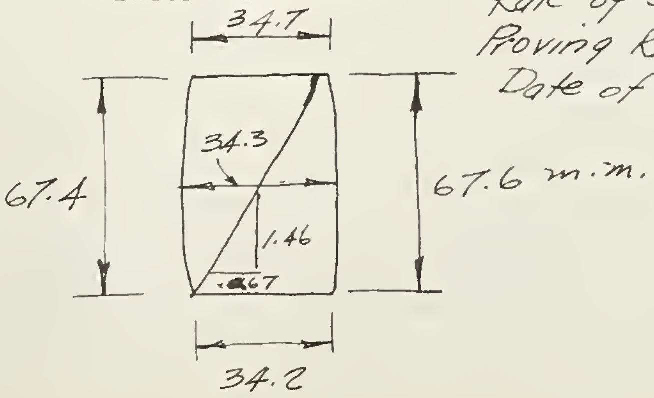
e Content – Initial 79.23%
Final 59.06 %

of Saturation – Initial 104% (?)
Final 96.7

tio – Initial 2.128
Final 1.703

essure Reaction .96

Sketch of Failure



Back Pressure 2 kg/cm²
Rate of Strain 408min/10m
Proving Ring 3-1498
Date of Test 28 July 61

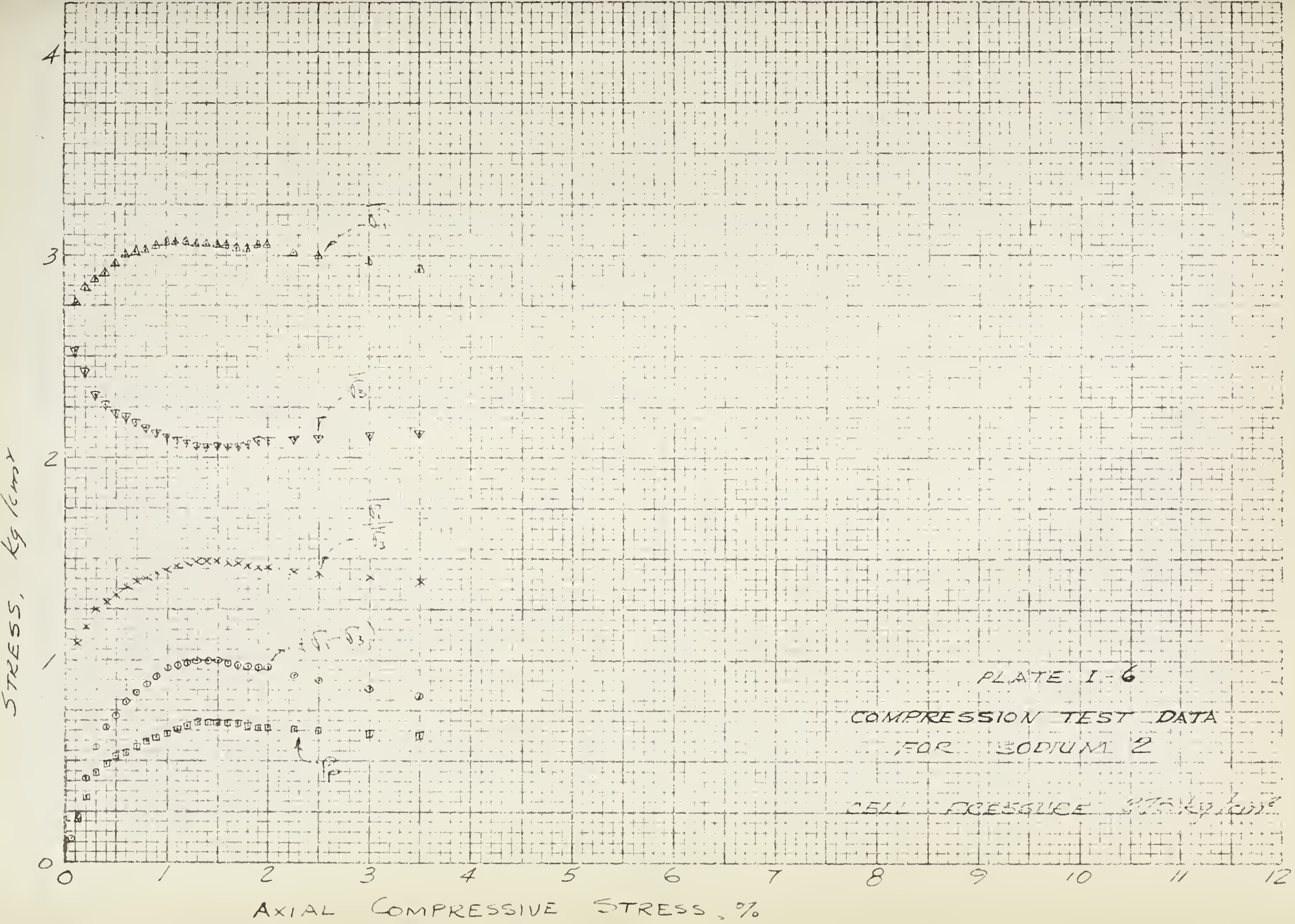


PLATE I-6
COMPRESSION TEST DATA
FOR SODIUM 2
CELL PRESSURE 3.75 kg/cm²
STRESS, kg/cm²
AXIAL COMPRESSIVE STRESS, %
0
1
2
3
4
5
6
7
8
9
10
11
12
$\overline{\sigma}_1$
$\overline{\sigma}_3$
$\overline{\sigma}_1 / \overline{\sigma}_3$
$\frac{1}{2}(\overline{\sigma}_1 - \overline{\sigma}_3)$
$\overline{p}$

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Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

mple Description CEC 32·9 me/100 gm
ldsorbed Cations: Ca 5·3%,
Na 87·9%, K 3·4, Mg 3·4 %
S.C. = 0

tial Weight 118·201 gms

Project THESIS
Hole No. SODIUM Gp 1
Depth ______ sample 3
Engineer ST
Technician DW
Date of Set-up 5 July 61
Test Lateral Pressure 4·0 kg/cm²

gth, mm	1. 76·28	2. 76·38	Aver: 76·33	Area Top	A_T 9.671	sq.mm
. mm Top	1. 35·04	2. 35·14	Aver. 35·09	Area Centre	A_C 9.948	sq.mm
Centre	1. 35·50	2. 35·68	Aver: 35·59	Area Bottom	A_B 10·139	sq.mm
Bottom	1. 35·96	2. 35·90	Aver: 35·93	Average X-Sect Area		

nal Volume 75·77 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$=$ 9·93

SOLIDATION DATA

e e	Δt min.	Burette Rdg. c.c.	ΔV c.c.
1040	0	24·00	
	1	23·78	
	2	23·74	
	4	23·65	
	6	23·60	
	8	23·55	
	10	23·50	
	15	23·43	
	20	23·35	
	25	23·30	
	30	23·25	
	60	23·00	
	100	22·77	
1310	150	22·55	
	250	22·22	
	300	22·10	
1720	400	21·85	
0830	1310	20·45	
✓	2750	19·05	
0900	4220	18·03	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
10 Jul 0900	7100	16·60	
11 ✓ 0840	8520	16·00	
12 ✓ ✓	9960	15.47	
13 ✓ ✓	11,400	14·97	
14 ✓ ✓	12,840	14·50	
15 ✓ 2215	15,095	13·81	
19 ✓ 0850	20,050	12·70	
22 ✓ 0945	24,425	11·84	
26 ✓ 1045	30,245	11·01	
31 ✓ 1310	37,590	10·30	
2 AUG 0845	40,205	10·09	
4 ✓ 0900	43,100	9.83	
7 ✓ 0820	47,380	9.58	
9 ✓ 0840	50,280	9.40	
11 ✓ 0935	53,215	9.30	
13 ✓ 1530	56,450	9.18	
14 ✓ 0830	57,470	9.16	

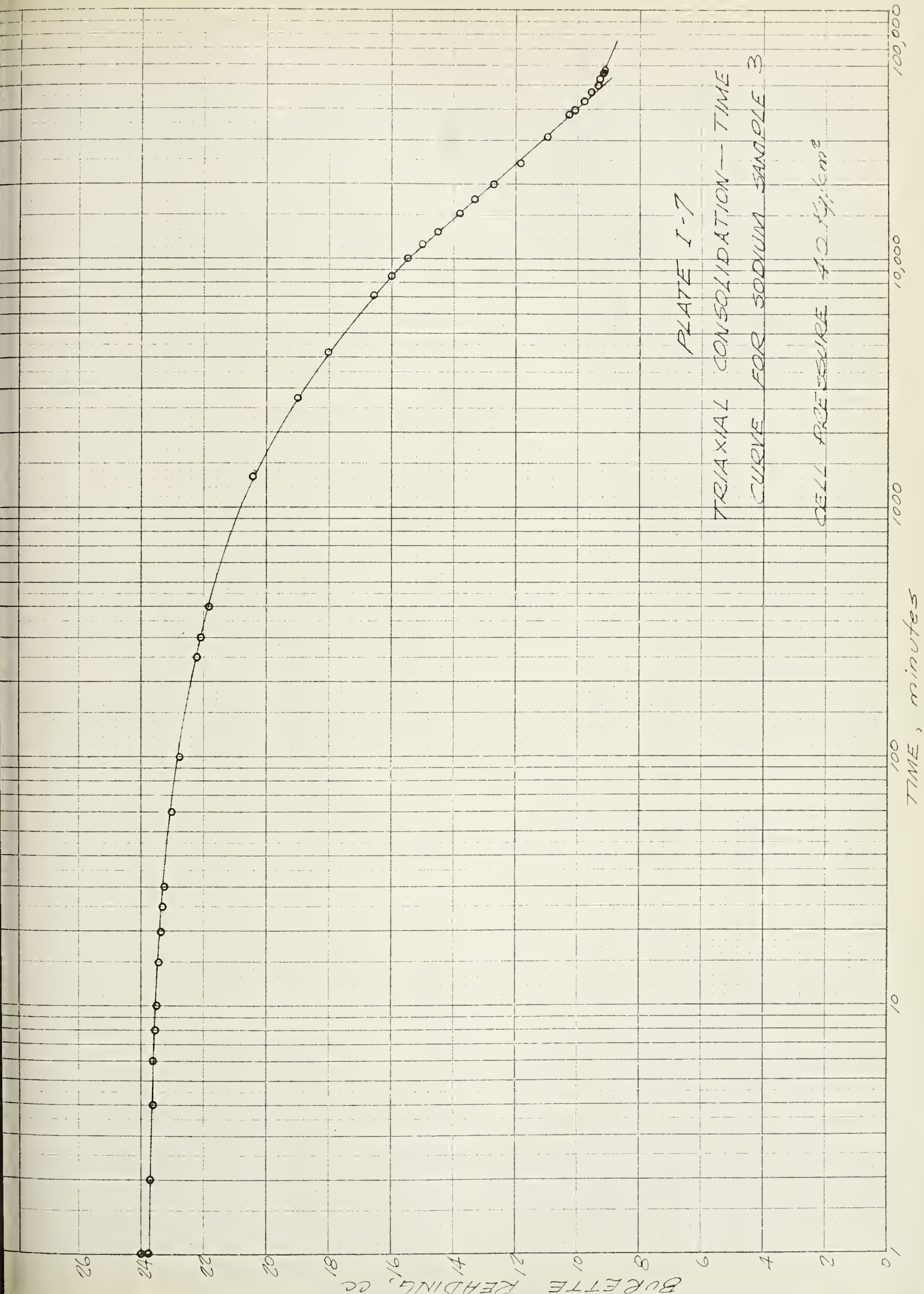


PLATE I-7

TRIAXIAL CONSOLIDATION — TIME CURVE FOR SODIUM SAMPLE 3

CELL PRESSURE 4.0 Kg/cm²

Sheet 1 I 17

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. SODIUM Gp 2

Depth Sample 3

Engineer JT

Technician DW

Date of Test 15 AUG 61

Test Lateral Pressure σ_3 4.0 kg/cm²

Back Pressure 2.0

Remarks Proving Ring 3-1498

Rate of Strain 552 min/10mm

Area Correction Factor 1.154

Failure

σ_3) = 1.37 σ_1 = 5.37

0.84

4.53

3.16

3.2 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_F	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\frac{\bar{A}\ P_P}{\sigma_1 - \sigma_3}$
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
0	9.93	0		0	+.02					
3	9.93	20	.0525	.12	+.05				.04	
5	9.93	40	.0511	.24	.07				.07	
7.5	9.94	57	.05025	.33	.08	4.25	3.92	1.08	0.1	24.2
15.5	9.95	107	.0484	.60	.18	4.42	3.82	1.16	.2	30.0
23	9.96	135	.04758	.74	.22	4.52	3.78	1.20	.3	29.7
30.5	9.97	161	.04676	.87	.28	4.59	3.72	1.23	.4	32.2
38	9.98	180	.04662	.97	.33	4.64	3.67	1.26	.5	34.0
46	9.99	194	.04638	1.04	.37	4.67	3.63	1.29	.6	35.6
53.5	10.00	208	.04616	1.11	.41	4.70	3.59	1.31	.7	36.9
61	10.01	219	.04600	1.16	.46	4.70	3.54	1.33	.8	39.7
68.5	10.02	227	.04589	1.20	.49	4.71	3.51	1.34	.9	40.8
76.5	10.03	235	.04578	1.24	.53	4.71	3.47	1.36	1.0	42.7
84	10.04	241	.04570	1.27	.54	4.73	3.46	1.367	1.1	42.5
91.5	10.05	247	.04562	1.29	.56	4.73	3.44	1.375	1.2	43.4
99	10.06	253	.04555	1.32	.58	4.74	3.42	1.385	1.3	43.9
107	10.07	258	.04550	1.35	.63	4.72	3.37	1.400	1.4	46.7
114.5	10.08	262	.04547	1.36	.64	4.72	3.36	1.405	1.5	47.1
122	10.09	264	.04543	1.37	.66	4.71	3.34	1.410	1.6	48.2
130	10.10	265	.04541	1.37	.66	4.71	3.34	1.410	1.7	48.2
137.5	10.11	266	.04540	1.38	.67	4.71	3.33	1.414	1.8	48.5
145	10.12	266	.04540	1.38	.70	4.68	3.30	1.418	1.9	50.7
152.5	10.13	267	.04539	1.38	.72	4.66	3.28	1.421	2.0	52.2
168	10.15	268	.04538	1.38	.73	4.65	3.27	1.422	2.2	52.9
183	10.175	269	.04537	1.38	.75	4.63	3.25	1.425	2.4	54.3
198.5	10.195	269	.04537	1.38	.79	4.59	3.21	1.430	2.6	57.2
213.5	10.215	269	.04537	1.38	.80	4.58	3.20	1.431	2.8	58.0
229	10.235	269	.04537	1.38	.81	4.57	3.19	1.433	3.0	59.7
244.5	10.26	268	.04538	1.37	.84	4.53	3.16	1.434	3.2	61.3

Sheet 2
I 18

UNIVERSITY OF ALBERTA

Department of Civil Engineering
Soil Mechanics Laboratory
AXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. SODIUM Gpl'
Depth
Sample 3
Area Correction Factor 1.154

e	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_F	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_P}{\sigma_1 - \sigma_3}$
7	259.5	10.28	265	.04541	1.35	.85	4.50	3.15	1.429	3.4	63.0
	275	10.30	260	.04547	1.32	.85	4.47	3.15	1.419	3.6	64.4
5	290	10.32	255	.04552	1.30	.84	4.46	3.16	1.411	3.8	64.6
3	305	10.345	250	.04558	1.27	.84	4.43	3.16	1.402	4.0	66.1
5	343.5	10.40	235	.04578	1.19	.84	4.35	3.16	1.376	4.5	70.6
8	381.5	10.45	226	.04590	1.15	.85	4.30	3.15	1.365	5.0	73.9

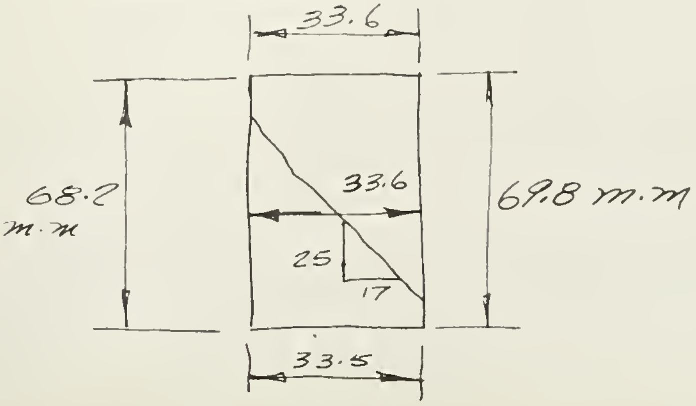
Content – Initial
Final 54.73

of Saturation – Initial 99.8
Final 97.8%

tio – Initial 2.188
Final 1.561

ssure Reaction 50%

Sketch of Failure



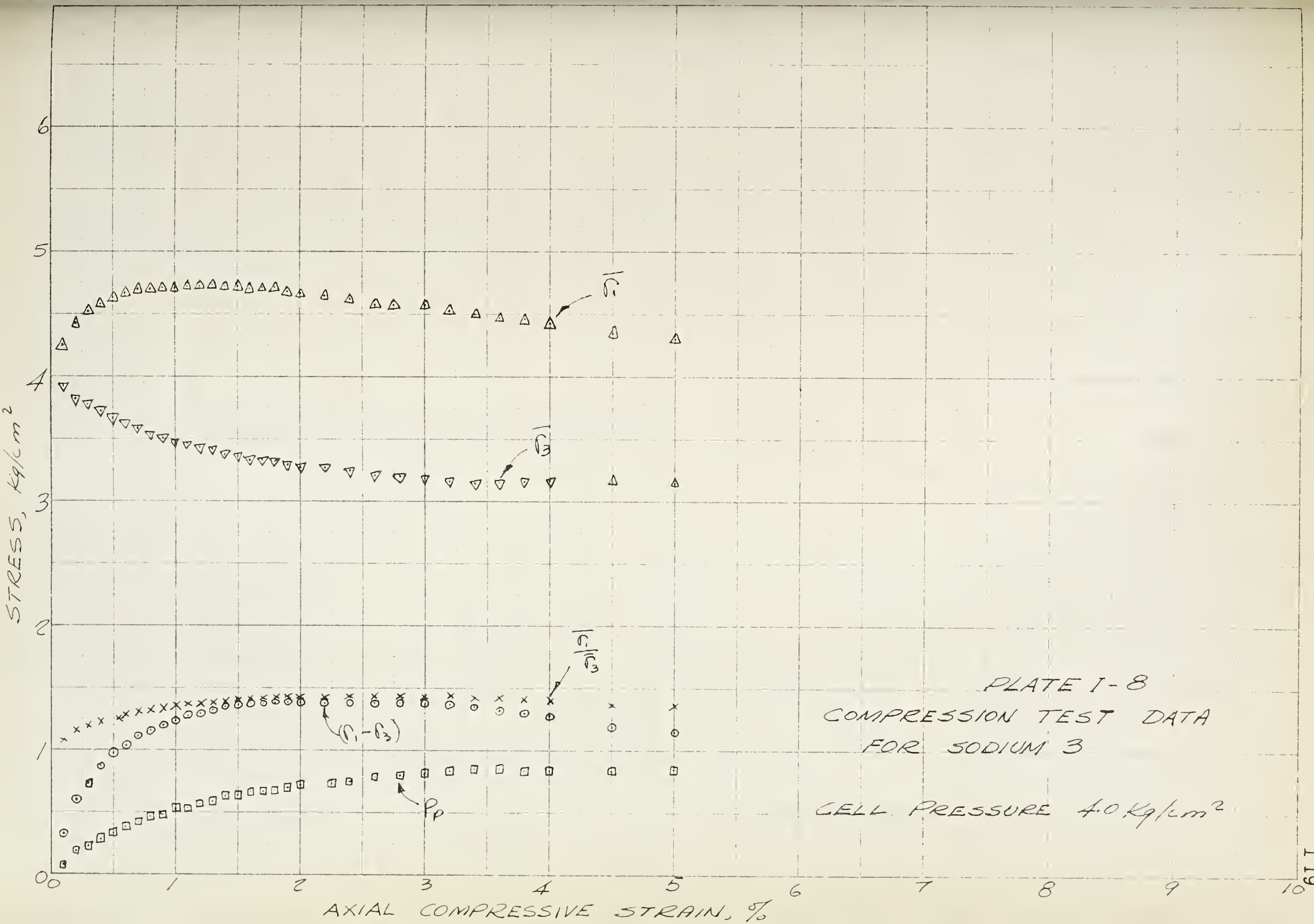


PLATE I-8
COMPRESSION TEST DATA
FOR SODIUM 3

CELL PRESSURE 4.0 Kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. SODIUM (Gp 2)
Depth sample 4
Engineer ST
Technician DW
Date of Set-up 19 SEP 61
Test Lateral Pressure 5.0 Kg/cm²

mple Description Cation Exchange
pacity: 32.9 me/100 gm soil.
erage adsorbed cations:
a 12.6 %, Na 80.7 %, K 3.6 %
Mg 3.2; Salt content pore H_2O = 3.8 m.e.
tial Weight 128.525 gms

gth, mm	1. 80.37	2. 80.37	Aver: 80.37	Area Top	A_T 9.80	sq.mm
. mm Top	1. 35.36	2. 35.29	Aver: 35.32	Area Centre	A_C 10.00	sq.mm
Centre	1. 35.66	2. 35.68	Aver: 35.67	Area Bottom	A_B 10.18	sq.mm
Bottom	1. 36.00	2. 36.00	Aver: 36.00	Average X-Sect Area		

nal Volume 80.37 cc

$= \frac{1}{4} (A_T + 2 A_C + A_B)$

$= 10.00$

SOLIDATION DATA

e e	Δt min.	Burette Rdg. c.c.	ΔV c.c.
61 1400	0	24.30	
	1	23.95	
	2	23.85	
	4	23.75	
	15	23.40	
	30	23.15	
	50	22.90	
	70	22.70	
	100	22.40	
	150	22.10	
1950	350	21.27	
2220	500	20.80	
0830	1110	19.47	
1400	1440	18.90	
0830	2550	17.50	
1630	3030	17.05	
0850	4010	16.20	
1200	5640	15.04	
1700	7380	14.03	
0820	8300	13.56	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
26 Sep 0920	9800	12.85	
27 〃 0815	11,175	12.25	
28 〃 1130	12,810	11.65	
29 〃 0915	14,115	11.20	
30 〃 1000	15,600	10.70	
2 Oct 0930		9.91	
3 Oct 1120	20,000	9.50	
4 Oct 0820		9.20	
5 〃 0930		8.89	
6 〃 0830		8.60	
7 Oct 0850	25,610	8.34	
9 Oct 2030	29,190	7.82	
13 〃 2220	35,060	7.12	
16 〃 0830	38,550	6.81	
17 〃 1050	44,450	6.70	
19 Oct 0910	47,230	6.50	
19 Oct 1340		6.50	

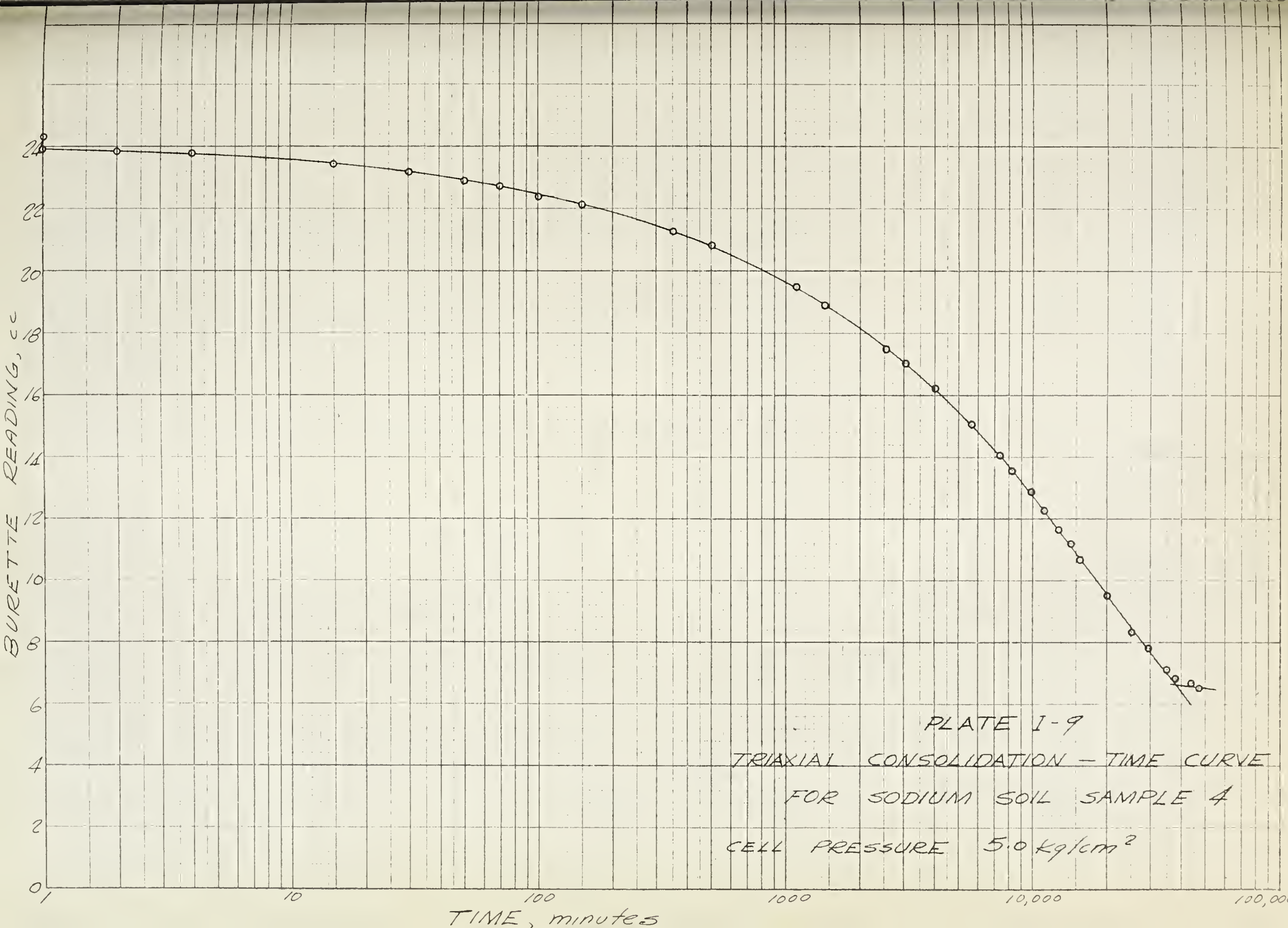


PLATE I-9

TRIAXIAL CONSOLIDATION – TIME CURVE

FOR SODIUM SOIL SAMPLE 4

CELL PRESSURE 5.0 kg/cm²

Sheet 1

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

t Failure

σ_3) = 2.11 σ_1= 7.11

1.48 Kg/cm²

5.63

3.52

6.5 %

Project THESIS

Hole No. SODIUM Gp 2

Depth Sample 4

Engineer ST

Technician DW

Date of Test 20 Oct 61

Test Lateral Pressure σ_3 5.0 kg/cm²

Back Pressure 2.0

Remarks Proving Ring 3-1498
Rate of Strain 362 min/10mm

Area Correction Factor K=1.188

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm² P_P	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\bar{A}$
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		$\frac{P_P}{\sigma_1-\sigma_3}$
0	10.00	0		0	0					
5	10.00	20	.0525	.105	.11					
11	10.00	40	.0511	.204	.11					
4.5	10.005	60	.0501	.30	.14				.056	
7	10.01	80	.0493	.47	.17				.087	
16	10.02	146	.04728	.82	.27	5.55	4.73	1.173	.2	32.9
24	10.03	184	.04652	1.01	.35	5.66	4.65	1.217	.3	34.7
32	10.04	218	.04600	1.19	.42	5.77	4.58	1.260	.4	35.3
40	10.05	242	.04570	1.31	.46	5.85	4.54	1.289	.5	35.1
48	10.06	264	.04542	1.42	.53	5.89	4.47	1.318	.6	37.3
56.5	10.07	284	.04520	1.51	.56	5.95	4.44	1.340	.7	37.1
64.5	10.08	297	.04507	1.58	.63	5.95	4.37	1.362	.8	39.9
72.5	10.09	308	.04498	1.63	.68	5.95	4.32	1.377	.9	41.7
80.5	10.10	320	.04487	1.69	.71	5.98	4.29	1.394	1.0	42.0
88.5	10.11	328	.04479	1.73	.73	6.00	4.27	1.405	1.1	42.2
96.5	10.12	338	.04470	1.77	.76	6.01	4.24	1.417	1.2	42.9
104.5	10.13	345	.04464	1.81	.79	6.02	4.21	1.430	1.3	43.6
112.5	10.14	351	.04460	1.83	.82	6.01	4.18	1.438	1.4	44.8
121	10.15	358	.04452	1.87	.85	6.02	4.15	1.451	1.5	45.5
129	10.16	364	.04450	1.89	.86	6.03	4.14	1.457	1.6	45.5
137	10.17	369	.04445	1.92	.88	6.04	4.12	1.466	1.7	45.8
145	10.18	374	.04440	1.94	.91	6.03	4.09	1.474	1.8	46.9
153	10.19	379	.04438	1.96	.93	6.03	4.07	1.482	1.9	47.4
161	10.20	383	.04433	1.98	.95	6.03	4.05	1.489	2.0	48.0
177	10.22	389	.04429	2.00	.98	6.02	4.02	1.498	2.2	49.0
193	10.24	396	.04423	2.03	1.04	5.99	3.96	1.513	2.4	51.2
209	10.26	400	.04421	2.05	1.07	5.98	3.93	1.522	2.6	52.2
225	10.29	405	.04419	2.07	1.10	5.97	3.90	1.531	2.8	53.1
241	10.31	409	.04415	2.08	1.13	5.95	3.87	1.537	3.0	54.3

Sheet 2

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. SODIUM (Gp 2)

Depth

Sample 4

Area Correction Factor 1.188

Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3$ $= \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm^2 P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1-\sigma_3}$
281	10.36	417	.04410	2.11	1.20	5.91	3.80	1.555	3.5	56.9
321.5	10.415	422	.04406	2.2	1.32	5.80	3.68	1.576	4.0	62.3
361.5	10.47	426	.04404	2.13	1.34	5.79	3.66	1.582	4.5	62.9
402	10.525	429	.04400	2.13	1.36	5.77	3.64	1.585	5.0	63.8
442	10.58	430	.04400	2.12	1.40	5.72	3.60	1.588	5.5	66.0
482	10.64	431	.04400	2.11	1.45	5.66	3.55	1.594	6.0	68.7
522.5	10.695	431	.04400	2.11	1.48	5.63	3.52	1.599	6.5	70.1
562.5	10.75	426	.04404	2.07	1.52	5.55	3.48	1.595	7.0	73.4
603	10.81	416	.04410	2.02	1.54	5.48	3.46	1.584	7.5	76.2
643	10.87	398	.04422	1.92	1.54	5.38	3.46	1.555	8.0	80.2
683	10.93	375	.04440	1.81	1.55	5.26	3.45	1.525	8.5	85.6

Content - Initial 68.3%

Final 46.23%

f Saturation - Initial 98.4%

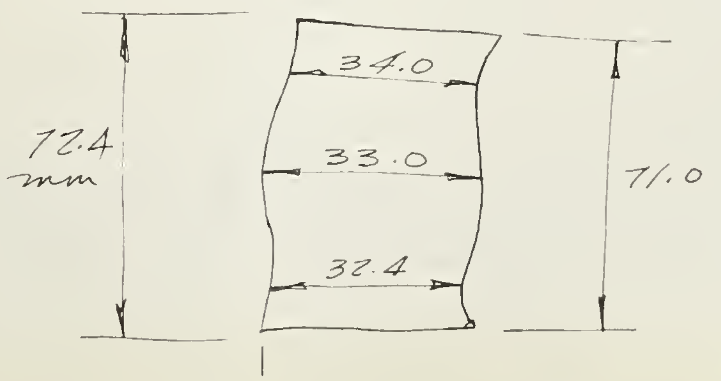
Final 97.4%

io - Initial 1.935

Final 1.324

ssure Reaction 50%

Sketch of Failure



No Signs of Shear

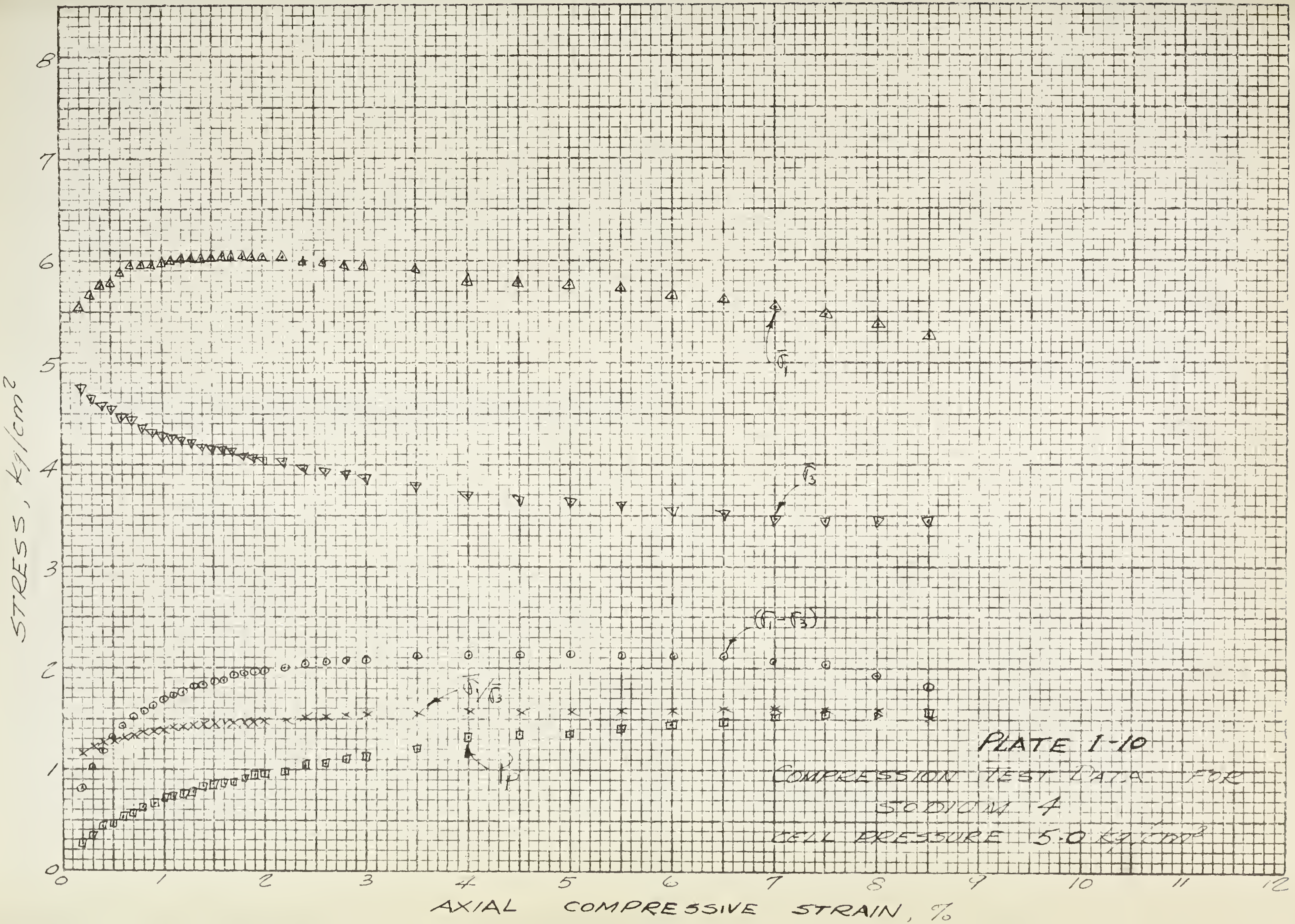


PLATE I-10

COMPRESSION TEST DATA FOR
SODIUM 4
CELL PRESSURE 5.0 kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

mple Description CEC 32.9 m.e/100gm
sorbed Cations : Ca 12.6 %
a 80.7 ; K: 3.6 %, Mg 3.2%
C. in pore Water: 3.8 me/100gm soil

tial Weight 127.247 gms

Project THESIS
Hole No. SODIUM (Gp 2)
Depth sample 5
Engineer ST
Technician -
Date of Set-up 25 Oct 61
Test Lateral Pressure 6.5 kg/cm²

th, mm	1. 80.30	2. 80.40	Aver: 80.35			
. mm Top	1. 35.24	2. 35.24	Aver. 35.24	Area Top	A_T	9.754 sq.mm
Centre	1. 35.60	2. 35.56	Aver: 35.58	Area Centre	A_C	9.943 sq.mm
Bottom	1. 36.00	2. 35.90	Aver: 35.95	Area Bottom	A_B	10.151 sq.mm

Average X-Sect Area
$= \frac{1}{4}(A_T + 2A_C + A_B)$
= 9.95

nal Volume 79.93 cc

SOLIDATION DATA

	Δt min.	Burette Rdg. c.c.	ΔV c.c.
0930	0	24.45	
	1	24.10	
	2	24.05	
	4	23.98	
	6	23.90	
	8	23.86	
	10	23.80	
	15	23.70	
	20	23.60	
	25	23.50	
	30	23.45	
	40	23.35	
	50	23.23	
	120	22.70	
1340	250	22.30	
1700	450	21.55	
305	815	20.50	
915	1425	19.45	
730	1920	18.80	
915	2865	17.60	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
28 Oct 0815	4245	16.27	
30 Oct 0815	7125	14.10	
1 Nov 0830	8580	12.30	
2 Nov 1045	10,155	11.45	
5 Nov 1510	14,740	9.30	
10 Nov 0930	21,600	7.10	
13 Nov 0830	25,860	6.20	
16 ✓ 1030	30,300	5.50	
20 ✓ 1130	36,120	4.80	
24 ✓ 1810	42,280	4.25	
28 ✓ 1200	47,670	3.95	
30 ✓ 1430	50,700	3.80	
4 Dec 1030	56 220	3.70	
11 Dec 0800	66,150	3.65	
27 Dec 1530	89640	3.60	20.85
		K=	1.22

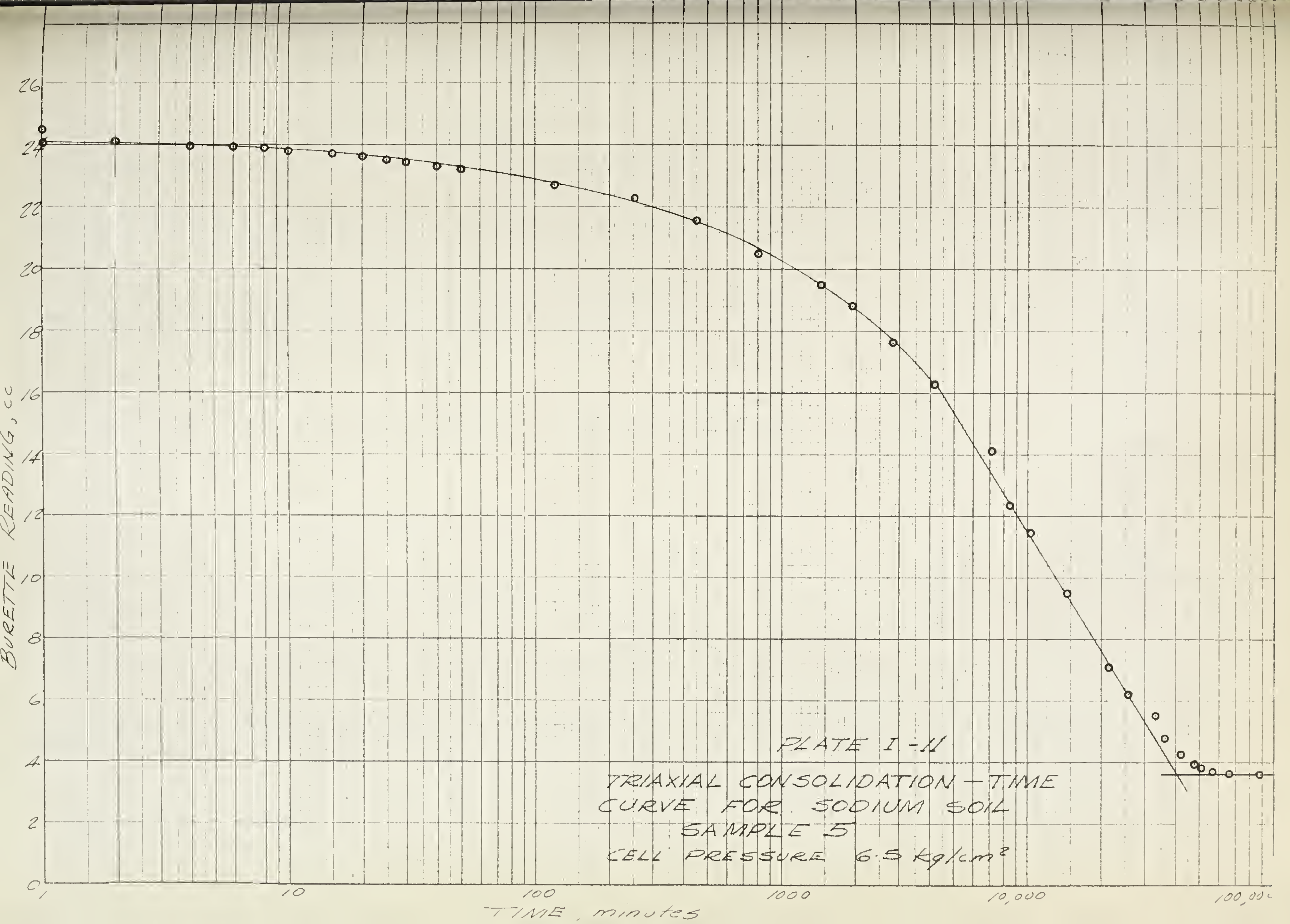


PLATE I - II

TRIAXIAL CONSOLIDATION - TIME CURVE FOR SODIUM SOIL
SAMPLE 5
CELL PRESSURE 6·5 kg/cm²

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

THESIS

SODIUM Gp 2

5

ST

-

28 DEC '61

σ_3 6.5 kg/cm²

2.0 " "

Proving Ring 3-1498

Rate of Strain 302.4 min/10mm

1.22

at Failure

σ_3 =

σ_1 =

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const. kg/Div	$\sigma_1 - \sigma_3$ kg/cm²	Press. kg/cm²					$\sigma_1 - \sigma_3$
0	9.95	0								
1	9.95	20	.0525							
5	9.95	40	.0511	.128	.11	6.52	6.39	1.020	.06	
7	9.96	60	.0501	.251	.16	6.59	6.34	1.039	.09	
9	9.96	80	.0493	.368	.18	6.69	6.32	1.058	0.11	48.9
10	9.96	100	.0486	.483	.19	6.79	6.31	1.076	0.12	39.6
16	9.97	180	.0466	.59	.28	6.81	6.22	1.095	.2	47.5
24	9.98	247	.04564	1.03	.37	7.16	6.13	1.168	.3	35.9
32	9.99	301	.04503	1.38	.50	7.38	6.00	1.230	.4	36.2
40	10.00	345	.04465	1.65	.61	7.54	5.89	1.280	.5	36.9
48	10.01	374	.04440	1.88	.70	7.68	5.80	1.324	.6	37.2
56	10.02	398	.04422	2.02	.79	7.73	5.71	1.364	.7	39.1
64	10.03	419	.04408	2.14	.88	7.76	5.62	1.381	.8	41.1
72	10.04	435	.04398	2.25	.95	7.80	5.55	1.405	.9	42.2
80	10.05	449	.04390	2.39	1.04	7.85	5.46	1.438	1.0	43.5
88	10.06	461	.04383	2.45	1.09	7.86	5.41	1.453	1.1	44.5
96	10.07	472	.04376	2.50	1.18	7.82	5.32	1.469	1.2	47.2
104	10.08	482	.04370	2.55	1.25	7.80	5.25	1.486	1.3	49.0
112	10.09	488	.04368	2.58	1.32	7.76	5.18	1.498	1.4	51.1
120	10.10	493	.04365	2.60	1.38	7.72	5.12	1.507	1.5	53.1
128	10.11	500	.04360	2.63	1.41	7.72	5.09	1.517	1.6	53.6
136	10.12	505	.04358	2.65	1.48	7.67	5.02	1.528	1.7	55.8
144	10.13	511	.04353	2.68	1.53	7.65	4.97	1.539	1.8	57.1
152	10.14	516	.04351	2.70	1.56	7.64	4.94	1.547	1.9	57.7
160.5	10.15	520	.04350	2.72	1.60	7.62	4.90	1.555	2.0	58.8
181	10.18	528	.04345	2.75	1.67	7.58	4.83	1.569	2.25	60.7
201	10.21	535	.04347	2.78	1.76	7.52	4.74	1.586	2.50	63.3
221	10.23	539	.04340	2.79	1.81	7.48	4.69	1.595	2.75	64.9
241	10.26	542	.04339	2.80	1.85	7.45	4.65	1.602	3.0	66.1

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. SODIUM (Gp 2

Depth Sample 5

Area Correction Factor 6.5 Kg/cm²

	Strain Dial Div.	A_c cm.	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm²	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A} = \frac{P_p}{\sigma_1 - \sigma_3}$
8	261	10.28	543	.04338	2.80	1.88	7.42	4.62	1.606	3.25	67.1
4	281	10.31	544	.04338	2.79	1.90	7.39	4.60	1.607	3.50	6.81.
1	301	10.34	544	.04338	2.78	1.94	7.34	4.56	1.610	3.75	69.7
7	321.5	10.36	543	.04338	2.77	1.96	7.31	4.54	1.610	4.00	70.7
55	341.5	10.39	542	.04339	2.76	1.97	7.29	4.53	1.609	4.25	71.4
1	361.5	10.42	540	.04340	2.74	1.98	7.26	4.52	1.606	4.5	72.2
8	381.5	10.45	538	.04340	2.73	1.99	7.24	4.51	1.605	4.75	73.0
4	402	10.47	536	.04342	2.71	1.99	7.22	4.51	1.601	5.0	73.4
7	442	10.53	530	.04343	2.67	1.98	7.19	4.52	1.590	5.5	74.1
0	482	10.585	522	.04349	2.62	1.97	7.15	4.53	1.578	6.0	75.2
3	522	10.64	512	.04353	2.55	1.96	7.09	4.54	1.562	6.5	76.8
7	562.5		523							7.0	

Content – Initial 71.2

Final 42.73

f Saturation – Initial 99.3

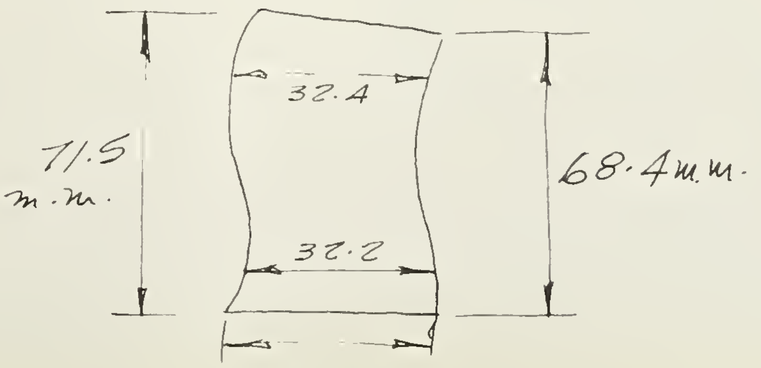
Final 97.3

io – Initial 2.00

Final 1.225

ssure Reaction 40%

Sketch of Failure



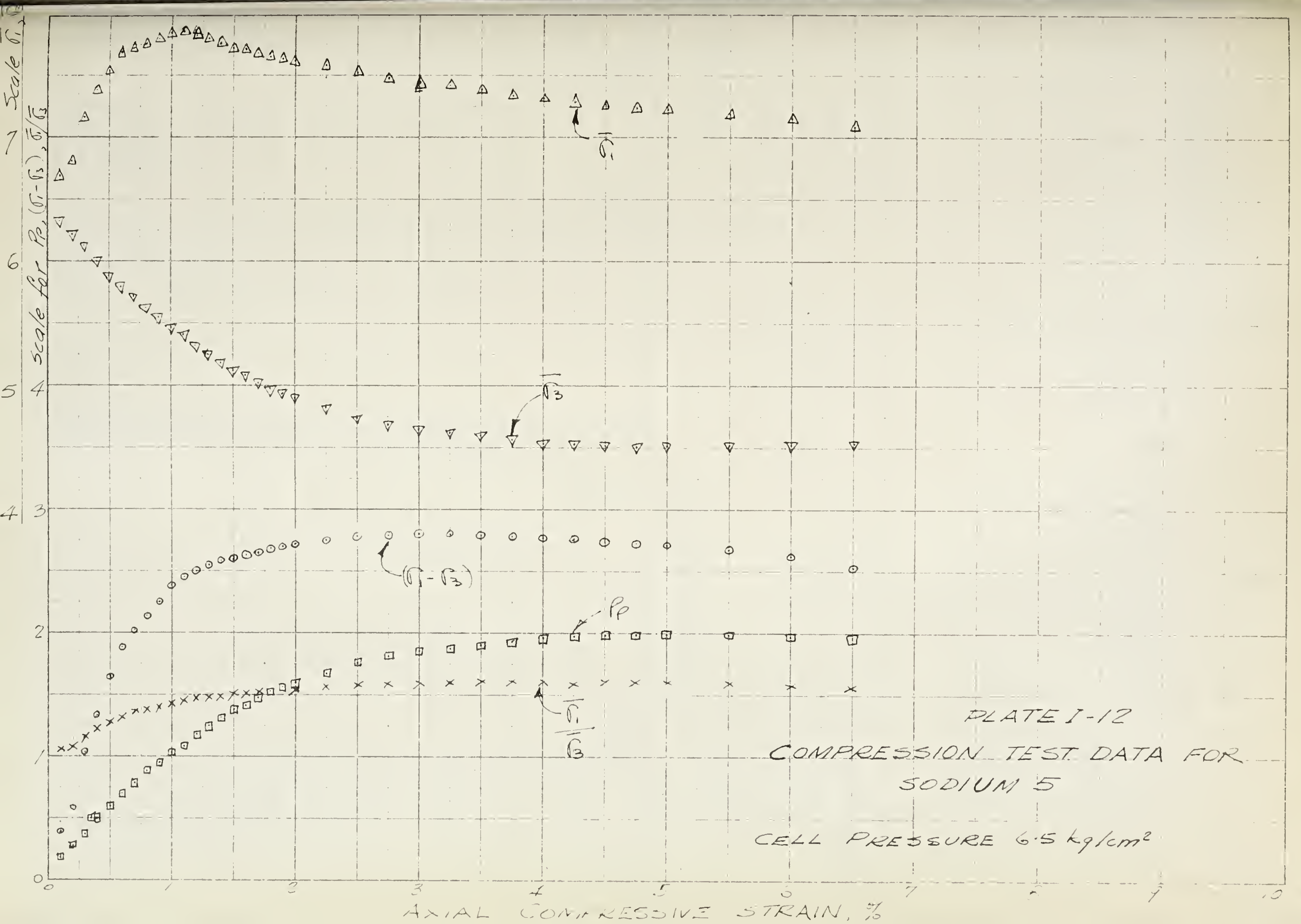


PLATE I-12

COMPRESSION TEST DATA FOR SODIUM 5

CELL PRESSURE 6·5 kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

IAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. SODIUM (GP2)
Depth sample 6
Engineer ST
Technician DW
Date of Set-up 4 AUG 1961
Test Lateral Pressure 7.5 Kg/cm²

nple Description CEC 32.9 me/100 gm Soil
sorbed Cations: Ca 12.6, Na 80.7
K 3.6 Mg 3.2; S.C in pore
ater 3.8 me/100 gm soil

ial Weight 128.850 gms

th, mm 1. 80.48 2. 80.46 Aver: 80.47
mm Top 1. 35.40 2. 35.30 Aver: 35.35
Centre 1. 35.78 2. 35.82 Aver: 35.80
Bottom 1. 36.08 2. 36.04 Aver: 36.06

Area Top A_T 9.815 sq.mm
Area Centre A_C 10.066 sq.mm
Area Bottom A_B 10.201 sq.mm
Average X-Sect Area
$= \frac{1}{4}(A_T + 2A_C + A_B)$
= 10.04

al Volume 80.77 cc

OLIDATION DATA

	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
0900	0	25.00	
	1	24.70	
	2	24.65	
	4	24.60	
	6	24.50	
	10	24.40	
	20	24.15	
	30	24.00	
	60	23.60	
	100	23.25	
	175	22.73	
	440	21.50	
1700	480	21.35	
0920	1460	18.40	
1130	3030	15.25	
8~~45~~20	4280	13.35	
845	5745	11.50	
835	7175	10.03	
845	8615	8.87	
145	10,065	7.83	

CONSOLIDATION DATA - cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
12 Aug 1145	11,685	6.90	
13 ✓ 1530	13,350	6.08	
15 ✓ 0820	15,800	5.02	
16 ✓ 0910	17,290	4.61	
18 ✓ 0900	20,160	3.97	
21 ✓ 0840	24,460	3.31	
22 ✓ 0900	25,920	3.11	
23 ✓ 0830	27,330	2.96	
25 ✓ 1000	30,300	2.70	
27 ✓ 2145	33,885	2.51	
29 ✓ 1220	36,200	2.40	
31 ✓ 1330	39,150	2.30	
3 Sep 1000	43,260	2.30	

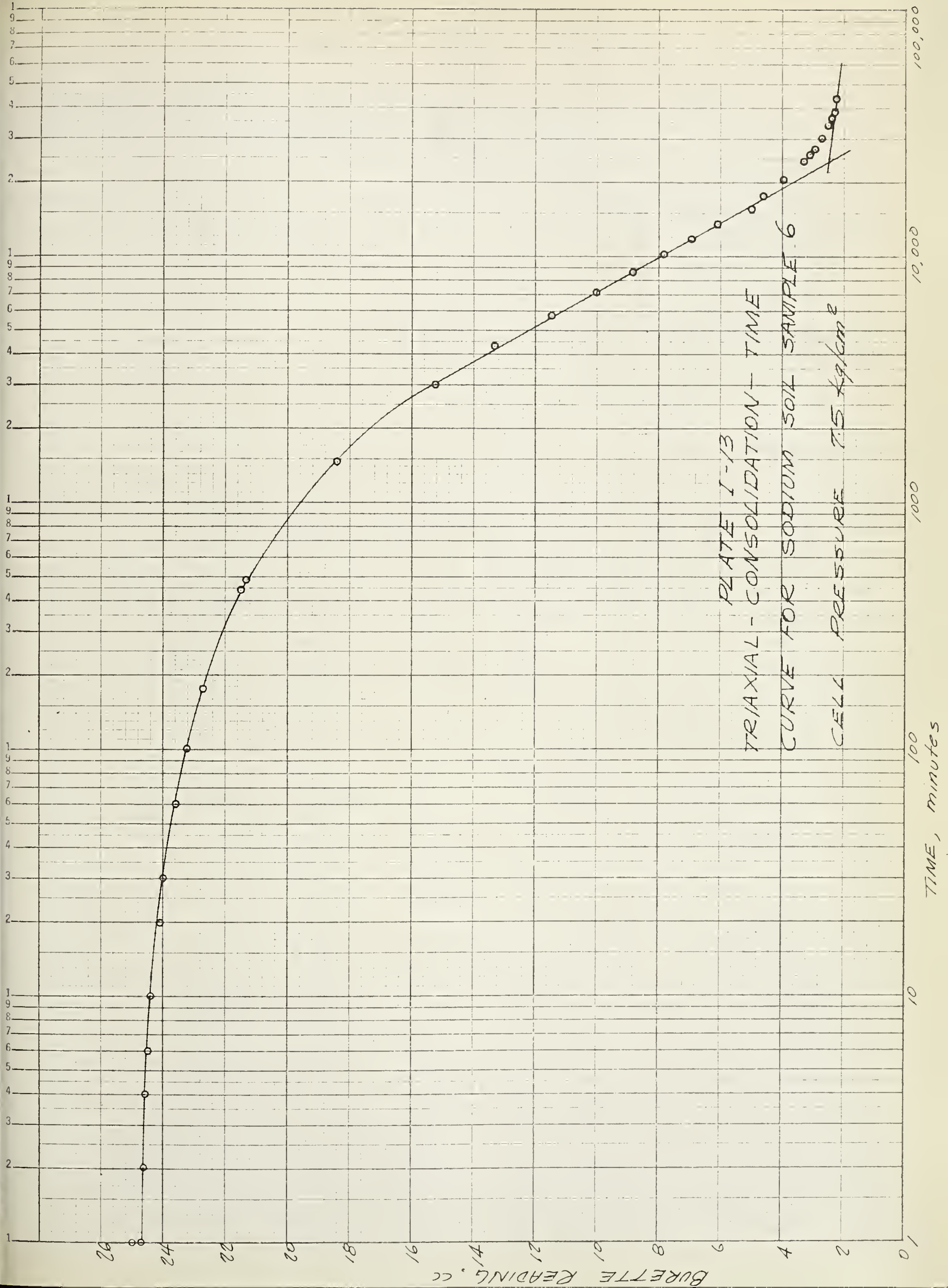


PLATE I-13

TRIAXIAL - CONSOLIDATION - TIME

CURVE FOR SODIUM SOIL SAMPLE 6

CELL PRESSURE 7.5 kg/cm²

Sheet 1

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. SODIUM (Gp 2)
Depth Sample 6
Engineer ST
Technician DW
Date of Test 4 Sep '61
Test Lateral Pressure σ_3 7.5 Kg/cm²
Back Pressure 2.0 " "
Remarks Proving Ring 3-1498
Rate of Strain 362·28 min/10 mm.

Area Correction Factor 1·25

t Failure
σ_3) = 3·13 σ_1 = 10·63
2·40 kg/cm²
8·23 "
5· 10
7·2 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\frac{\bar{A}}{P_p}$ $\sigma_1 - \sigma_3$
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
0	10.04	0		0	+.16					
1.5	10.04	~~05~~20	.0525	.13	+.20				.02	
3	10.04	40	.0511	.25	.24				.04	
4	10.04	60	.0501	.37	.28				.05	
6	10.045	80	.0493	.49	.32				.075	
8	10.05	118	.04800	.70	.37	7.83	7.13	1.10	.10	52.8
16	10.06	216	.04603	1.24	.58	8.16	6.92	1.18	.2	46.7
24	10.07	290	.04516	1.63	.76	8.37	6.74	1.24	.3	46.6
32	10.08	343	.04467	1.90	.88	8.52	6.62	1.29	.4	46.3
40	10.09	383	.04435	2.10	1.00	8.60	6.50	1.32	.5	47.6
48.5	10.10	416	.04414	2.27	1.11	8.66	6.39	1.36	.6	48.9
56.5	10.11	438	.04396	2.38	1.18	8.70	6.32	1.38	.7	49.6
645	10.12	457	.04386	2.48	1.24	8.74	6.26	1.40	.8	50.0
72.5	10.13	475	.04374	2.56	1.28	8.78	6.22	1.41	.9	50.0
80.5	10.14	489	.04366	2.63	1.34	8.79	6.16	1.43	1.00	50.9
88.5	10.15	503	.04359	2.70	1.41	8.79	6.07	1.44	1.10	52.7
96.5	10.16	515	.04351	2.76	1.45	8.81	6.05	1.46	1.20	52.5
104.5	10.17	524	.04347	2.80	1.50	8.80	6.00	1.47	1.30	53.6
112.5	10.18	532	.04343	2.84	1.52	8.82	5.78	1.475	1.40	53.5
120.5	10.19	539	.04340	2.87	1.57	8.80	5.93	1.484	1.50	54.7
129	10.20	546	.04338	2.90	1.60	8.80	5.90	1.492	1.60	55.4
137	10.21	554	04333	2.94	1.63	8.81	5.87	1.501	1.70	55.4
145	10.22	559	.04330	2.96	1.66	8.80	5.84	1.507	1.80	56.1
153	10.23	564	.04329	2.98	1.68	8.80	5.82	1.512	1.90	56.4
161	10.245	570	.04327	3.01	1.71	8.80	5.79	1.520	2.00	56.8
177	10.265	578	.04322	3.04	1.79	8.75	5.71	1.532	2.2	58.9
193	10.285	585	.04320	3.07	1.83	8.74	5.61	1.541	2.4	59.6
209	10.31	591	.04317	3.09	1.87	8.72	5.63	1.549	2.6	60.5
225.5	10.33	597	.04314	3.12	1.91	8.71	5.59	1.558	2.8	61.2

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. SODIUM Gp 2

Depth ______ Sample 6

Area Correction Factor 1.25

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_P	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\frac{\bar{A}\ P_P}{\sigma_1 - \sigma_3}$
						$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\bar{\sigma}_1/\bar{\sigma}_3$		
241.5	10.35	602	.04312	3.14	1.94	8.70	5.56	1.565	3.0	61.8
257.5	10.37	606	.04310	3.15	1.98	8.67	5.52	1.571	3.2	62.9
273.5	10.39	611	.04309	3.17	2.00	8.67	5.50	1.576	3.4	63.1
289.5	10.41	614	.04308	3.18	2.04	8.64	5.46	1.582	3.6	64.2
306	10.435	618	.04306	3.19	2.07	8.62	5.43	1.587	3.8	64.9
322	10.46	621	.04305	3.19	2.10	8.59	5.40	1.591	4.0	65.8
338	10.48	623	.04304	3.20	2.12	8.58	5.38	1.595	4.2	66.3
354	10.50	625	.04303	3.20	2.14	8.56	5.36	1.597	4.4	66.9
370	10.525	626	.04303	3.20	2.18	8.52	5.32	1.602	4.6	68.1
386.5	10.55	628	.04302	3.20	2.22	8.48	5.28	1.606	4.8	69.4
402.5	10.57	629	.04301	3.20	2.24	8.46	5.26	1.608	5.0	70.0
418.5	10.59	630	.04301	3.20	2.26	8.44	5.24	1.611	5.2	70.6
434.5	10.61	631	.04301	3.20	2.28	8.42	5.22	1.613	5.4	71.3
450.5	10.63	631	.04301	3.19	2.31	8.38	5.19	1.615*	5.6	72.4
466.5	10.66	631	.04301	3.18	2.29	8.39	5.21	1.610	5.8	72.0
483	10.68	631	.04301	3.18	2.29	8.39	5.21	1.610	6.0	72.0
499	10.705	631	.04301	3.17	2.31	8.36	5.19	1.611	6.2	72.9
515	10.725	631	.04301	3.16	2.33	8.33	5.17	1.611	6.4	73.7
531	10.75	630	.04301	3.15	2.35	8.30	5.15	1.612	6.6	74.6
547	10.77	630	.04301	3.14	2.33	8.31	5.17	1.607	6.8	74.2
563.5	10.79	630	.04301	3.14	2.38	8.26	5.12	1.613	7.0	75.8
579.5	10.82	629	.04301	3.13	2.40	8.23	5.10	1.614	7.2	76.7 ←
595.5	10.84	627	.04302	3.11	2.43	8.18	5.07	1.613	7.4	78.7
611.5	10.86	625	.04303	3.09	2.45	8.14	5.05	1.612	7.6	79.3
627.5	10.89	623	.04304	3.08	2.44	8.14	5.06	1.609	7.8	79.2
644	10.91	620	.04305	3.06	2.47	8.09	5.03	1.608	8.0	80.7

Cell pressure crept up (c/c piston at top of trave) ∴ Pp likely a little high

∴ Ratio $\bar{\sigma}_1/\bar{\sigma}_3$ also too high ∴ not taken as failure.

Content – Initial 68.9

Final 39.51

f Saturation – Initial 98.3

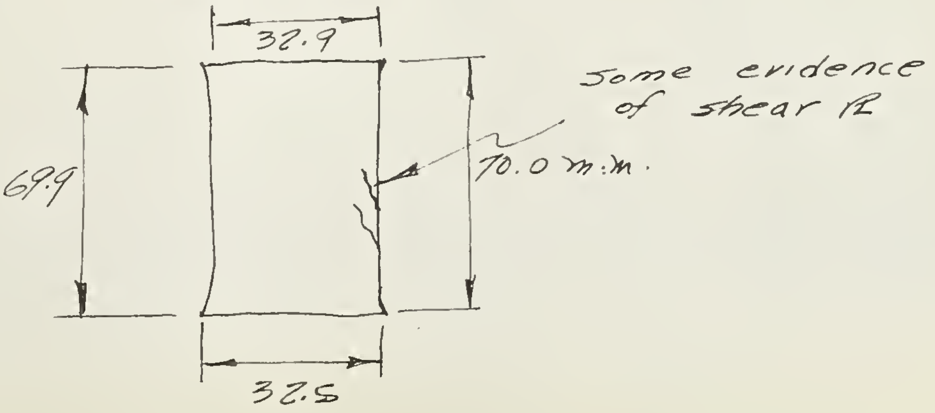
Final 96.0

io – Initial 1.951

Final 1.148

ssure Reaction 75%

Sketch of Failure



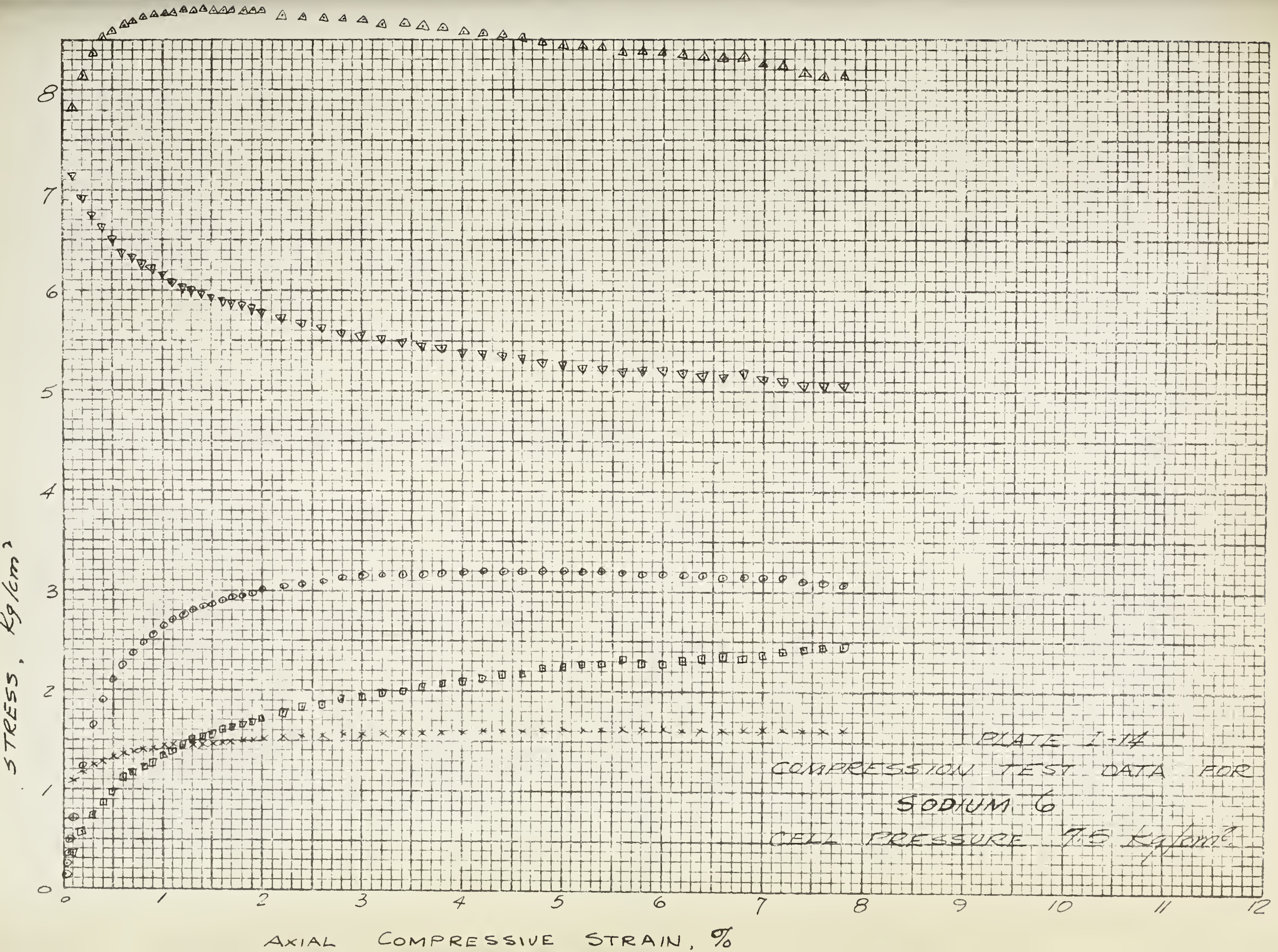


PLATE I-14
COMPRESSION TEST DATA FOR
SODIUM 6
CELL PRESSURE 7.5 Kg/cm²

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. SODIUM (high Salts)

Depth sample 7

Engineer ST

Technician NS

Date of Set-up 7 JAN 1962

Test Lateral Pressure 4.0 Kg/cm²

mple Description Prepared by shaking
mains of previous samples
 2N NaAc. No washing after
Ac.

tial Weight 134.088 gms

th, mm	1. 80.56	2. 80.52	Aver: 80.54	Area Top	A_T 9.848	Sq.mm
. mm Top	1. 35.42	2. 35.40	Aver. 35.41	Area Centre	A_C 10.010	sq.mm
Centre	1. 35.72	2. 35.68	Aver: 35.70	Area Bottom	A_B 10.134	sq.mm
Bottom	1. 35.90	2. 35.94	Aver: 35.92	Average X-Sect Area		

nal Volume 80.54 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$=$ 10.00

SOLIDATION DATA

	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
0930	0	24.75	
	.1	24.20	
	.25	24.05	
	.50	23.90	
	1.0	23.65	
	2	23.30	
	3	22.95	
	4	22.60	
	6	22.10	
	8	21.70	
	10	21.35	
	15	20.45	
	20	19.70	
	25	19.10	
	30	18.50	
	35	18.00	
	40	17.55	
	45	17.15	
	50	16.75	
	60	16.00	

CONSOLIDATION DATA - cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
8 JAN 1040	70	15.50	
	80	15.00	
	100	14.30	
	126	13.45	
	150	12.80	
	180	12.25	
1305	215	11.70	
1350	260	11.20	
1500	330	10.65	
1610	400	10.30	
1750	500	10.00	
2325	835	9.55	
9 JAN 0945	1455	9.45	
" 1330	1680	9.45	
10 1600	3270	9.25	

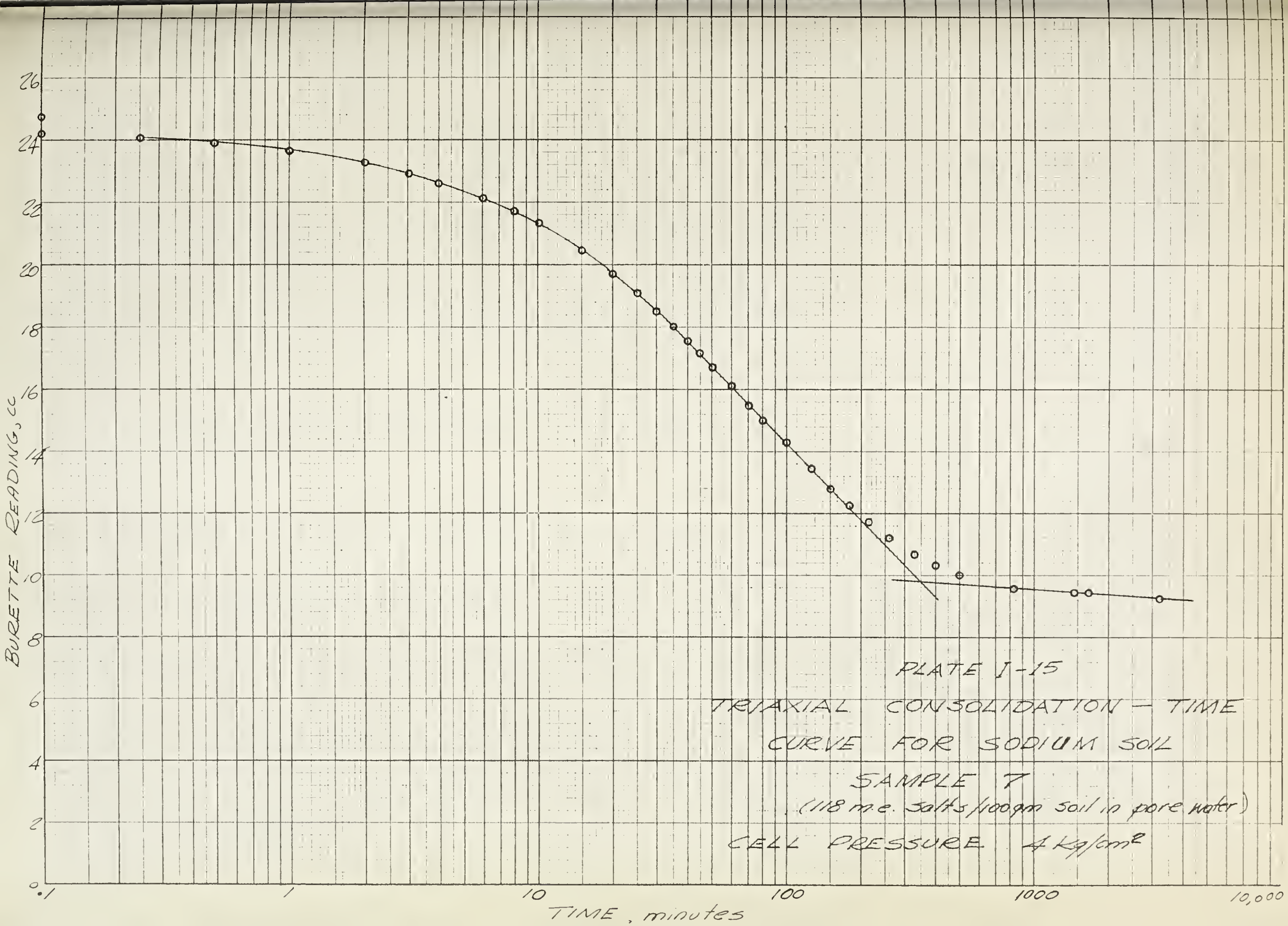


PLATE I-15

TRIAXIAL CONSOLIDATION – TIME
CURVE FOR SODIUM SOIL

SAMPLE 7
(118 m.e. salts/100gm soil in pore water)

CELL PRESSURE 4 Kg/cm²

Sheet 1

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. SODIUM (high salts
Depth — Sample 7
Engineer ST
Technician
Date of Test 11 JAN 62
Test Lateral Pressure σ3 4.0 Kg/cm²
Back Pressure 2. " "
Remarks Proving Ring 3-1498
Rate of Strain 302min/10mm
Area Correction Factor 1.16

Failure
σ3) = 2.75 σ1= 6.75
2.57 Kg/cm²
4.18
1.43
10.05 %

Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot h}{A_c}$	Pore Press kg/cm² Pp	Effective Stress $\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_p}{\sigma_1-\sigma_3}$
	10.00	0	0	0	+.12					
1		20	.0525	.105	.28					
2		40	.0511	.204	.32					
4		60	.0501	.306	.35				.05	
8	10.01	115	.0481	.64	.48	4.16	3.52	1.182	.10	
16	10.02	213	.04608	1.14	.72	4.42	3.28	1.348	.2	63.2
24	10.03	268	.04539	1.41	.89	4.52	3.11	1.453	.3	63.1
32	10.04	310	.04495	1.61	1.04	4.57	2.96	1.544	.4	64.6
40	10.05	342	.04462	1.76	1.13	4.63	2.87	1.613	.5	
48	10.06	363	.04450	1.90	1.25	4.65	2.75	1.691	.6	65.8
56	10.07	383	.04433	1.96	1.34	4.62	2.66	1.737	.7	
64	10.08	400	.04421	2.03	1.43	4.60	2.57	1.790	.8	70.4
72	10.09	413	.04412	2.09	1.50	4.59	2.50	1.836	.9	71.8
80.5	10.10	425	.04404	2.15	1.56	4.59	2.44	1.881	1.0	72.6
88.5	10.11	436	.04398	2.20	1.60	4.60	2.40	1.917	1.1	72.7
96.5	10.12	446	.04390	2.24	1.66	4.58	2.34	1.957	1.2	74.1
105	10.13	455	.04386	2.28	1.72	4.56	2.28	2.000	1.3	
113	10.14	460	.04383	2.31	1.74	4.57	2.26	2.022	1.4	
121	10.15	468	.04379	2.34	1.79	4.55	2.21	2.059	1.5	76.5
129	10.16	475	.04374	2.37	1.82	4.55	2.18	2.087	1.6	
137	10.17	481	4370	2.40	1.88	4.52	2.12	2.132	1.7	78.3
147	10.18	487	.04368	2.42	1.89	4.53	2.11	2.147	1.8	
153	10.19	491	.04366	2.44	1.93	4.51	2.07	2.179	1.9	
161	10.20	497	.04360	2.46	1.97	4.49	2.03	2.212	2.0	80.1
181	10.23	506	.04357	2.50	2.03	4.47	1.97	2.269	2.25	81.2
201	10.26	514	.04352	2.53	2.08	4.45	1.92	2.318	2.50	82.2
221.5	10.28	521	.04349	2.56	2.14	4.42	1.86	2.376	2.75	83.6
241.5	10.31	527	.04375	2.58	2.18	4.40	1.82	2.418	3.0	84.5
282	10.36	537	.04340	2.61	2.24	4.37	1.71	2.483	3.5	85.8

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

AXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. SODIUM (high salts)

Depth

Sample 7

Area Correction Factor 1.16

	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1 - \sigma_3}$
48	322	10.42	543.5	.04338	2.62	2.29	4.33	1.71	2.532	4.0	87.4
62	362.5	10.47	548.5	.04336	2.63	2.33	4.30	1.67	2.575	4.5	88.6
5	403	10.53	552	.04334	2.64	2.38	4.26	1.62	2.630	5.0	90.2
8	443	10.58	555	.04333	2.64	2.42	4.22	1.58	2.671	5.5	91.6
2	483	10.64	557	.04331	2.63	2.43	4.20	1.57	2.675	6.0	92.4
16	523.5	10.70	564	.04329	2.65	2.46	4.19	1.54	2.721	6.5	92.8
30	564	10.75	577.5	.04323	2.69	2.51	4.18	1.49	2.805	7.0	93.3
44	604	10.81	587	.04320	2.72	2.52	4.20	1.48	2.838	7.5	92.6
58	644	10.87	593	.04317	2.73	2.54	4.19	1.46	2.870	8.0	93.0
72	685	10.93	600	.04313	2.75	2.54	4.21	1.46	2.884	8.5	92.4
87	730	11.00	606	.04310	2.75	2.56	4.19	1.44	2.910	9.06	93.1
01	771	11.06	609	.04310	2.75	2.56	4.19	1.44	2.910	9.57	93.1
13	809	11.12	611	.04310	2.75	2.57	4.48	1.45	2.923	10.05	93.5
40	896	11.25	601	.04313	2.67	2.57	4.10	1.43	2.867	11.12	96.3
48	926	11.30	576	.04323	2.56	2.56	4.00	1.44	2.778	11.5	100
54	954	11.35	507	.04357	2.26	2.51	3.75	1.49	2.511	11.87	111.0

e Content – Initial 55.3%

Final 36.2

of Saturation – Initial 98.0

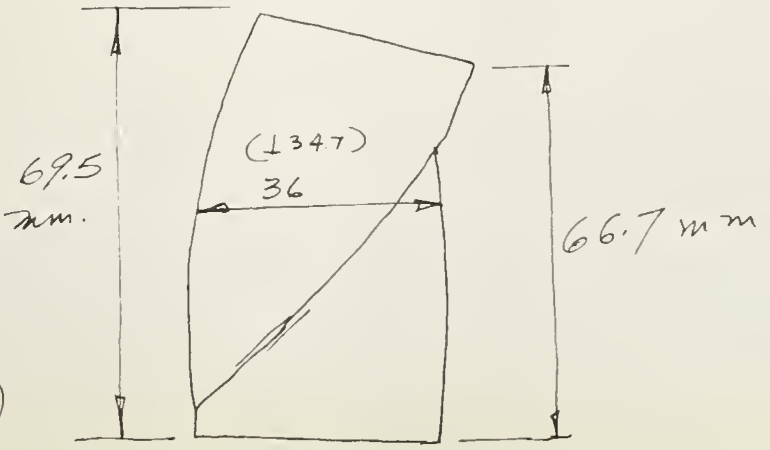
Final 86

tio – Initial 1.54

Final 1.15

essure Reaction 0.96 (or 96%)

Sketch of Failure



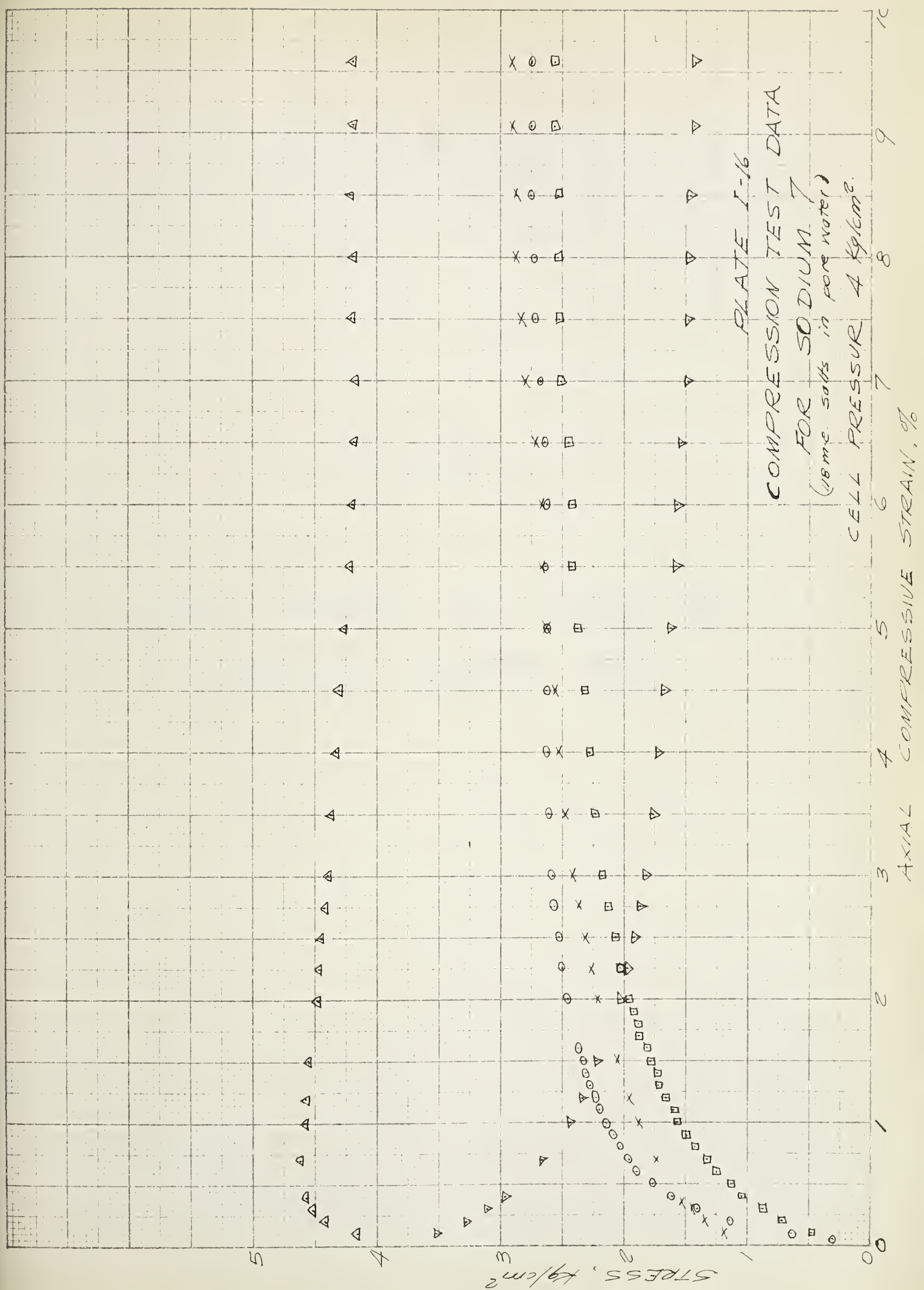


PLATE I-16

COMPRESSION TEST DATA FOR SODIUM 7 (18 me. salts in pore water)

CELL PRESSURE 4 Kg/cm².

APPENDIX J

DATA SHEETS FOR TESTS ON MAGNESIUM MODIFIED SOIL

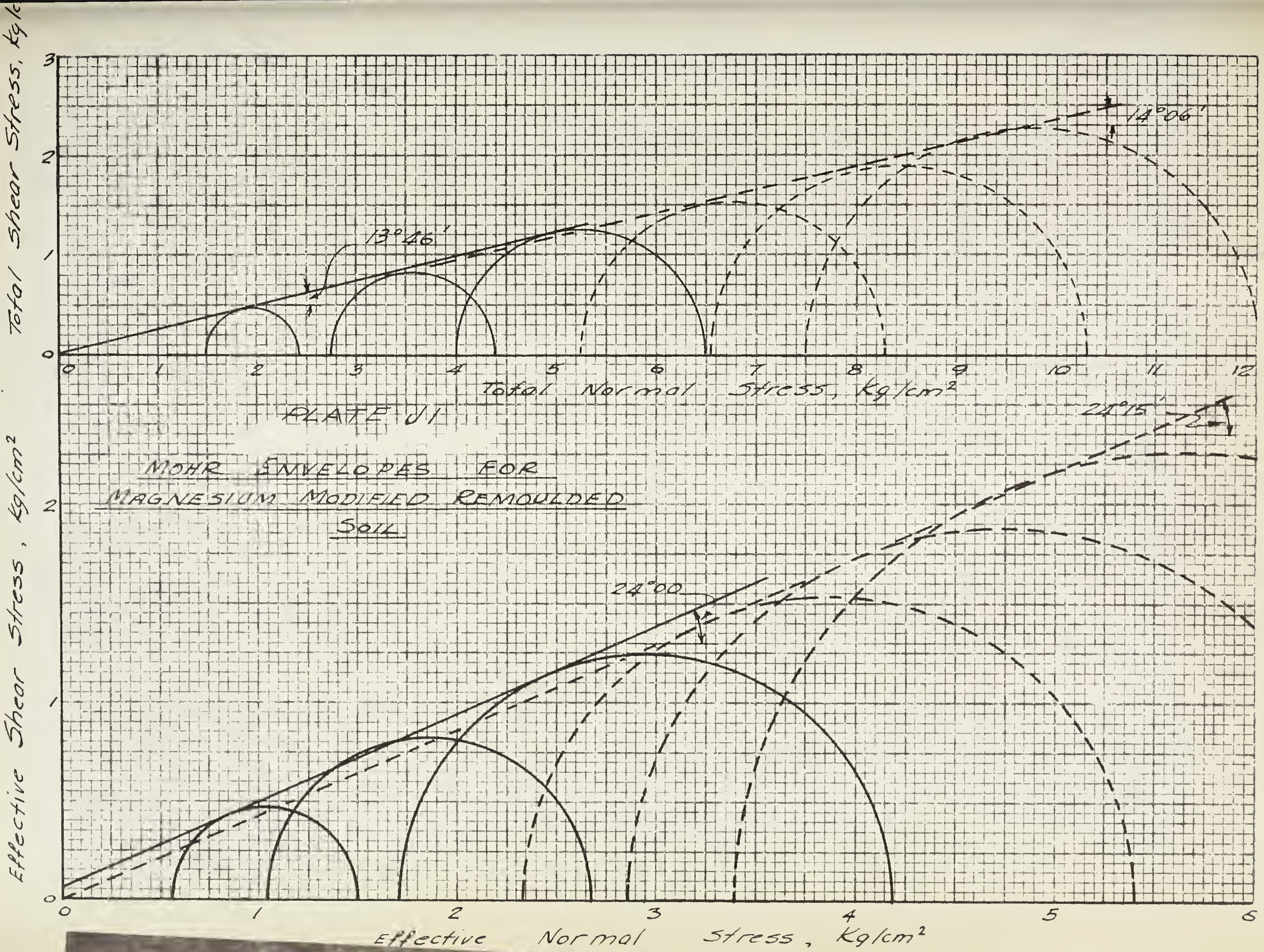


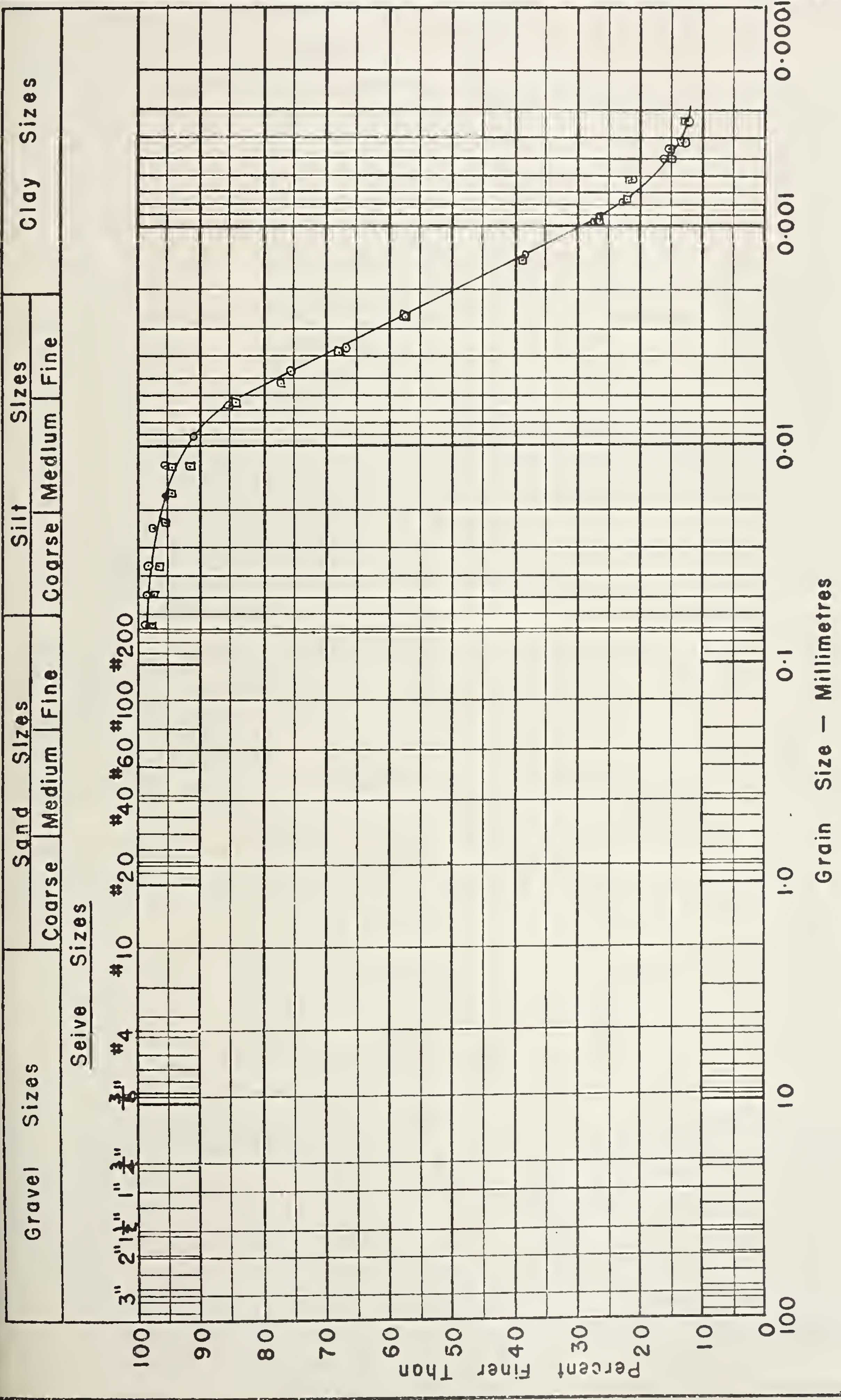
PLATE J1

MOHR ENVELOPES FOR MAGNESIUM MODIFIED REMOULDED SOIL

UNIVERSITY of ALBERTA
DEPT. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

GRAIN SIZE CURVE

PROJECT	THESIS		
SITE			
SAMPLE	MAGNESIUM		
LOCATION			
HOLE		DEPTH	
TECHNICIAN	DW&ST	DATE	21 July 61



Legend
⊙ Sample 1
△ Sample 2

D₁₀ = ____ mm.
D₆₀ = ____ mm.
Cu ____

Note: M.I.T. Grain Size Scale

Remarks: From Hamilton's Work:
W_L 79.2 %
W_p 30.1 %
I_p 49.1 %
Compressive Index 0.63 ; Activity 1

PLATE J2

UNIVERSITY of ALBERTA
DEP'T. of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

HYDROMETER TEST

PROJECT THESIS
SITE
SAMPLE MAGNESIUM #2 (Gpl)
LOCATION
HOLE
DEPTH
TECHNICIAN DW
DATE 13 July 61

Date	Temp. °C	Time	Elapsed Time	R_h'	$R_h = R_h' + c_m$	D m.m.	$R_h + m_t - c_d$	W. %	W % Basis Orig. Sample	Remarks
13 July		1012	0							
			0.5^{m}	19.8	20.3	.068	20.8	98.2		
			1	19.7	20.2	.049	20.7	97.7		
			2	19.5	20.0	.035	20.5	96.7		
			4	19.3	19.8	.023	20.3	95.8		
	24.2		8	19.0	19.5	.017	20.0	94.4		
			15	19.6	19.5	.012	20.0	94.4		
	24.2		30	18.5	19.0	.012	19.5	92.0		
	24.1		60	17.0	17.5	.0066	18.0	85.0		
	23.9		$1^{h}:45^{m}$	15.3	15.8	.0051	16.3	76.9		
	23.8		$3^{h}:15^{m}$	13.5	14.0	.0038	14.5	68.4		
	23.8		$6^{h}\,25^{m}$	11.2	11.7	.0027	12.2	57.6		
14 July	24.1	0917	23:05	7.1 5.0	7.6 5.5	.0015 .00094	8.1 6.0	38.2 28.3		
15 ✓	23.4	2130	59:18	4.9 3.7	5.4 4.2	.00096 .00061	5.8 4.6	27.4 21.7		
17 ✓	21.7	0830	94:18	4.4 2.8	4.9 3.3	0008 0005	4.9 3.2	23.1 15.1		
18 ✓	22.4	1100	120:48	2.5	3.0	.00045	3.2	15.1		
19 ✓	22.4	1030	144:18	2.2	2.7	.00041	2.9	13.7		
20 ✓	22.8	1137	169:25	2.1	2.6	.00034	2.8	13.2		

Hydrometer No.s 9857R and 50700 R Graduate No.

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s - 1} (R_h + m_t - c_d) = $ 4.72 $(R_h + m_t - c_d)$

Meniscus correction = c_m = 0.5 and 0.5 respectively

Dispersing agent used Calgon Amount 10 ml

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = 0.3 and 0.3 respectively

Specific Gravity of Solids = G_s 2.68

Description of Sample

Method of Preparation

Remarks

Initial Moisture Content

Container No.
Wt. Sample Wet + Tare
Wt. Sample Dry + Tare
Wt. Water
Tare Container
Wt. of Dry Soil
Initial Moisture w %

Dry Weight of Sample

Container No.
Wt. Sample (Wet/~~Dry~~) + Tare 1046.4
Tare 1012.6
Wt. (~~Wet~~/Dry) Soil 33.8
Dry Weight from Initial Moisture = $\frac{100 \times \text{Wt. Wet Soil}}{100 + \text{Init. Moist.}\%}$ =

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DEP'T· of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

HYDROMETER TEST

PROJECT THESIS
SITE
SAMPLE MAGNESIUM 1 (Gp 1)
LOCATION
HOLE DEPTH
TECHNICIAN DW DATE 13Jul61

Date	Temp· °C	Time	Elapsed Time	R'_h	$R_h = R'_h + c_m$	D m.m.	$R_h + m_t - c_d$	W·%	W % Basis Orig· Sample	Remarks
13July		0955	0							
			.5m	19.9	20.4	.068	21.0	99.1		
			1	19.8	20.3	.049	20.9	98.6		
			2	19.8	20.3	.035	20.9	98.6		
			4	19.6	20.1	.024	20.7	97.7		
	24.3		8	19.1	19.6	.018	20.2	95.3		
			15	19.1	19.6	.013	20.2	95.3		
	24.3		30	18.3	18.8	.0093	19.4	91.6		
	24.1		60	16.9	17.4	.0065	17.9	84.5		
	23.9		2h:00m	15.0	15.5	.0047	16.0	75.5		
	23.8		3:30	13.3	13.8	.0037	14.3	67.5		
	23.8		6:40	11.2	11.7	.0026	12.2	57.6		
14July	24.1	0915	23:20	7.2 / 5.0	7.7 / 5.5	.00145 / .00094	8.2 / 6.0	38.7 / 28.3		
15 ✓	23.3	2130	59:35	5.1 / 3.7	5.6 / 4.2	.00096 / .00061	5.9 / 4.5	27.8 / 21.2		
17 ✓	21.6	0830	94:35	4.4 / 3.0	4.9 / 3.5	.00079 / .00050	4.9 / 3.5	23.1 / 16.5		
18 ✓	22.4	1100	121:05	2.5	3.0	.00045	3.2	15.1		
19 ✓	22.5	1030	144:35	2.0	2.5	.00041	2.7	12.7		
20 ✓	22.8	1135	169:40	2.0	2.5	.00034	2.7	12.7		

Hydrometer No.s 9857 R and 50700 R Graduate No. ______

$W\% = \frac{100}{W_s} \cdot \frac{S_s}{S_s - 1} (R_h + m_t - c_d) = 4.72 \, (R_h + m_t - c_d)$

Meniscus correction = c_m = 0.5 and 0.5 respectively

Dispersing agent used Calgon Amount 10 ml

Correction for change in density of liquid due to addition of dispersing agent = c_d

c_d = 0.3 and 0.3 respectively

Specific Gravity of Solids = G_s 2.68

Description of Sample ______

Method of Preparation ______

Remarks ______

Initial Moisture Content

Container No. ______
Wt· Sample Wet + Tare ______
Wt· Sample Dry + Tare ______
Wt· Water ______
Tare Container ______
Wt· of Dry Soil ______
Initial Moisture w % ______

Dry Weight of Sample

Container No. ______
Wt. Sample (Wet/~~Dry~~) + Tare 1121.6
Tare 1087.8
Wt· (~~Wet~~/Dry) Soil 33.8
Dry Weight from Initial
Moisture = $\frac{100 \times \text{Wt· Wet Soil}}{100 + \text{Init· Moist.}\%}$ = ______

UNIVERSITY of ALBERTA
DEP'T· of CIVIL ENGINEERING
SOIL MECHANICS LABORATORY

SPECIFIC GRAVITY

PROJECT *THESIS*
SITE
SAMPLE *MAGNESIUM*
LOCATION
HOLE DEPTH
TECHNICIAN *DW & ST* DATE *As Noted*

Sample No.	1 29Jun61	2 13Jul61	3 13Jul61	
Flask No.	101	101	101	
Method of Air Removal	Vacuum	Vacuum	Vacuum	Vacuum
W_{b+w+s}	692.02	691.40	693.93	
Temperature T °C	22.7	28.0	22.8	
W_{b+w}	667.59	666.93	668.69	
Evaporating Dish No.				
Wt. Sample Dry + Dish	225.96	230.97	211.33	
Tare Dish	186.95	191.89	171.32	
W_s	39.01	39.08	40.01	
G_s	2.68	2.67	2.68	

W_{b+w+s} = Weight of flask + water + sample at T°.
W_{b+w} = Weight of flask + water at T° (flask calibration curve).
W_s = Weight of dry soil
G_s = Specific gravity of soil particles $= \frac{W_s}{W_s + W_{b+w} - W_{b+w+s}}$

Determination of W_s from wet soil sample:

Sample No.			Sample No.		
Container No.			Container No.		
Wt. Sample Wet + Tare			Wt. Test Sample Wet + Tare		
Wt. Sample Dry + Tare			Tare Container		
Wt. Water			Wt. Test Sample Wet		
Tare Container			W_s		
Wt. of Dry Soil					
Moisture Content w %					

Description of Sample:

Remarks:

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Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Sample Description Cation Exchange Capacity 32·6 me/100 gm soil, Average Cations adsorbed Ca 9·0%, Na 0·6, K 3·1%, Mg 87·3. Salt content of pore water 1·8 me/100gm soil

Initial Weight 130·545 gms

Project THESIS
Hole No. MAGNESIUM Gp 1
Depth
Sample 1
Engineer ST
Technician DW
Date of Set-up 19 JUN 61
Test Lateral Pressure 1·5 Kg/cm²

ngth, mm	1. 78·34	2. 78·24	Aver: 78·29	Area Top A_T 10·139 sq.mm
am. mm Top	1. 35·86	2. 36·00	Aver. 35·93	Area Centre A_C 9·954 sq.mm
Centre	1. 35·60	2. 35·60	Aver: 35·60	Area Bottom A_B 9·709 sq.mm
Bottom	1. 35·20	2. 35·12	Aver: 35·16	Average X-Sect Area

iginal Volume 77·81 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

= 9·94

NSOLIDATION DATA

ate me	Δt min.	Burette Rdg. c.c.	ΔV c.c.
N 1335	0	24·65	
	·1	24·30	
	·25	24·20	
	·5	24·10	
	1	23·90	
	2	23·70	
	4	23·30	
	6	23·00	
	8	22·75	
	10	22·50	
	15	22·00	
	20	21·60	
	30	21·00	
	45	20·35	
	61	19·85	
	100	18·95	
	200	17·75	
1925	350	17·10	
2130	475	16·85	
N 0840	1145	16·50	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
21 Jun 0840	2585	16·40	
1610	3035	16·40	8·25
		K = 1·075	

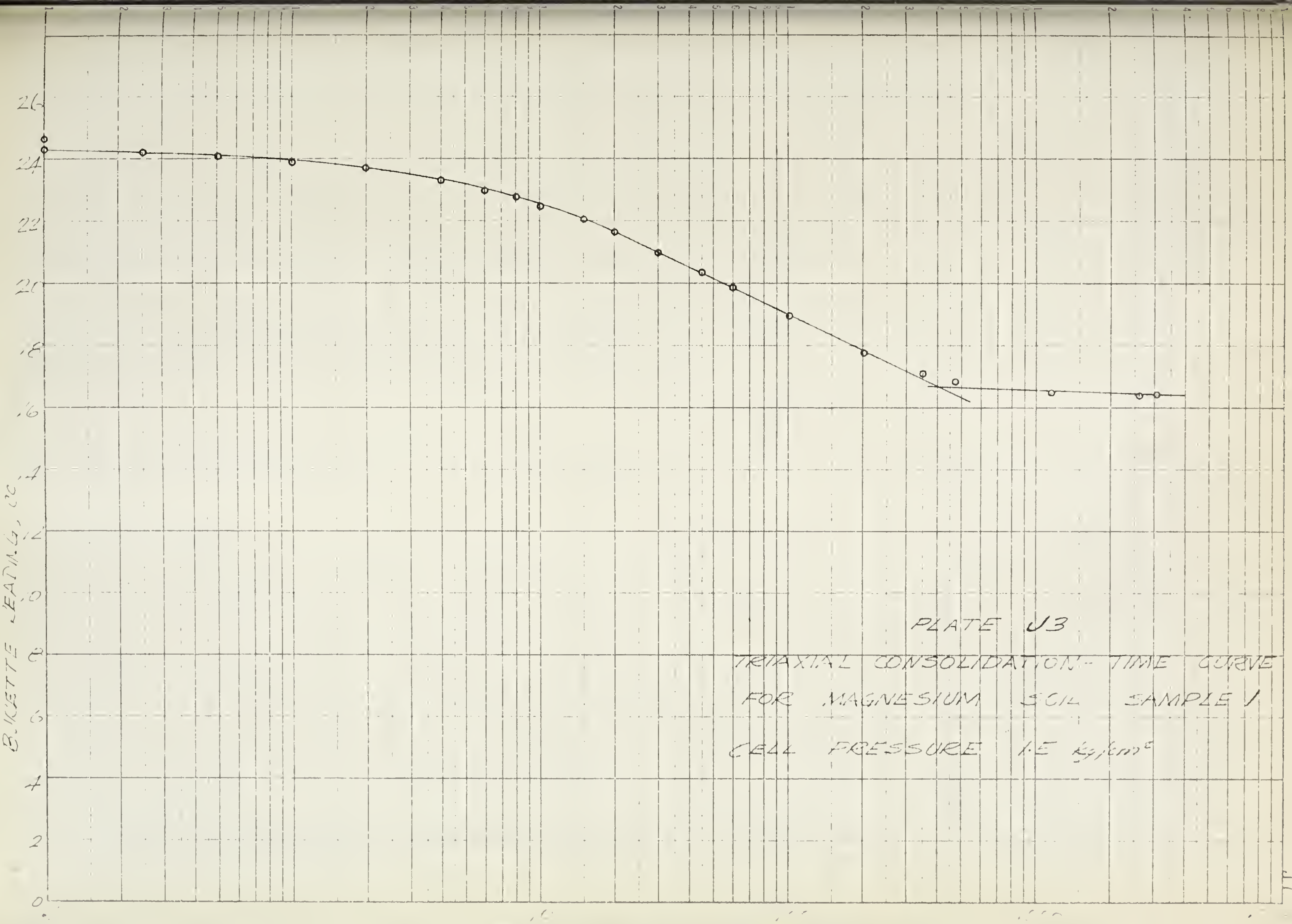

PLATE J3

TRIAXIAL CONSOLIDATION - TIME CURVE
FOR MAGNESIUM SOIL SAMPLE 1
CELL PRESSURE 1.5 kg/cm²

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Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. MAGNESIUM Gp 1
Depth Sample 1
Engineer ST
Technician DW
Date of Test 22 JUN 61
Test Lateral Pressure σ_3 1·50 kg/cm²
Back Pressure 2 kg/cm²
Remarks Proving Ring 3-1498
Rate of Strain 552 min/10 mm

at Failure
$- \sigma_3) =$ 0·93 $\sigma_1 =$ 2·43
0·93 Kg/cm²
1·50
0·57
7·2 %

Area Correction Factor 1·075

ne n 00	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pt	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_p}{\sigma_1 - \sigma_3}$
0	0	9·94			0	+·05					
4	1·5	✓	20	·0525	·11	+·06				·02	
5	3	✓	30	·0517	·17	·07				·04	
3	8	9·95	64	·04995	·35	·13	1·72	1·37	1·26	·1	37·1
3	16	9·96	99	·04865	·52	·27	1·75	1·23	1·42	·2	51·9
1	24	9·97	119	·0480	·62	·32	1·80	1·18	1·53	·3	51·6
7	31·5	9·98	131	·04768	·67	·38	1·79	1·12	1·60	·4	56·7
3	39·5	9·99	139	·04747	·71	·42	1·79	1·08	1·66	·5	59·2
8	47	10·00	146	·04730	·74	·44	1·80	1·06	1·70	·6	59·5
4	55	10·01	151	·04720	·77	·49	1·78	1·01	1·76	·7	63·6
9	63	10·02	155	·04710	·78	·50	1·78	1·00	1·78	·8	64·1
4	70·5	10·03	158	·04700	·80	·51	1·79	0·99	1·81	·9	63·7
0	78·5	10·04	162	·04695	·81	·53	1·78	0·97	1·84	1·0	65·4
5	86	10·05	165	·04688	·83	·56	1·77	·94	1·88	1·10	67·5
0	94	10·06	167	·04685	·84	·58	1·76	·92	1·91	1·20	69·0
5	102	10·07	170	·04679	·85	·59	1·76	·91	1·93	1·30	69·4
9	109·5	10·08	172	·04675	·86	·61	1·75	·89	1·97	1·40	70·9
4	117·5	10·08	173	·04674	·86	·62	1·74	·88	1·98	1·50	72·1
0	125·5	10·10	175	·04670	·87	·64	1·73	·86	2·01	1·60	73·6
4	133	10·11	177	·04665	·88	·65	1·73	·85	2·04	1·70	73·8
8	141	10·12	178	·04664	·88	·66	1·72	·84	2·05	1·80	75·0
4	149	10·13	180	·04660	·89	·68	1·71	·82	2·09	1·90	76·4
8	156·5	10·14	181	·04658	·87	·69	1·70	·81	2·10	2·00	77·5
7	172·5	10·15	183	·04655	·90	·70	1·70	·80	2·13	2·20	77·8
7	188	10·17	185	·04652	·91	·71	1·70	·79	2·15	2·40	78·0
7	203·5	10·19	186	·04650	·91	·74	1·67	·76	2·20	2·60	81·3
6	~~70~~ 219	10·22	187	·04648	·71	·75	1·66	·75	2·21	2·8	82·4
5	235	10·24	188	·04647	·72	·77	1·65	·73	2·26	3·0	83·7
5	250·5	10·26	184	·04645	·92	·79	1·63	·71	2·30	3·2	85·7

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Sheet 2
J9

Project THESIS

Hole No. MAGNESIUM

Depth Sample 1

Area Correction Factor 1.075

ime in	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm² P_F	Effective Stress $\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\,P_F}{\sigma_1-\sigma_3}$
184	266	10.28	191	.04643	.93	.80	1.63	.70	2.33	3.40	86.0
94	282	10.30	191	.04643	.93	.81	1.62	.69	2.35	3.60	87.1
03	297.5	10.315	192.5	.04640	.93	.82	1.61	.68	2.37	3.8	88.2
13	313	10.335	193	.04640	.93	.82	1.61	.68	2.37	4.0	88.2
21	329	10.36	194	.04637	.93	.83	1.60	.67	2.39	4.2	89.2
30	344.5	10.38	195	.04636	.94	.84	1.60	.66	2.42	4.4	89.3
41	360	10.41	196	.04635	.94	.85	1.59	.65	2.45	4.6	90.4
50	376	10.43	196	.04635	.94	.85	1.59	.65	2.45	4.8	90.4
60	391.5	10.45	196	.04635	.93	.87	1.56	.63	2.48	5.0	93.5
68	407	10.47	197	.04633	.94	.88	1.56	.62	2.52	5.2	93.6
278	423	10.49	197	.04633	.94	.88	1.56	.62	2.52	5.4	93.6
87	438.5	10.52	197	.04632	.94	.88	1.56	.62	2.52	5.6	93.6
96	454	10.53	198	.04632	.94	.89	1.55	.61	2.54	5.8	94.6
06	470	10.56	198	.04632	.93	.89	1.54	.61	2.52	6.0	95.6
15	486	10.58	198	.04632	.93	.90	1.53	.60	2.55	6.2	96.8
26	501	10.60	198	.04632	.93	.90	1.53	.60	2.55	6.4	96.8
35	517	10.62	198	.04632	.93	.91	1.52	.59	2.58	6.6	97.8
44	532.5	10.64	198	✓	.93	.92	1.51	.58	2.60	6.8	98.9
54	548	10.66	199	.04630	.93	.93	1.50	.57	2.63	7.0	100.0
63	564	10.69	199	.04630	.93	.93	1.50	.57	2.63	7.2	100
73	579.5	10.71	199	.04630	.92	.93	1.49	.57	2.61	7.4	101.1
82	595	10.73	199	.0430	.92	.94	1.48	.56	2.64	7.6	102.1
01	626.5	10.78	199	.04630	.92	.95	1.47	.55	2.67	8.0	103.3

ure Content – Initial 53.75

Final 46.17

e of Saturation – Initial 98.9

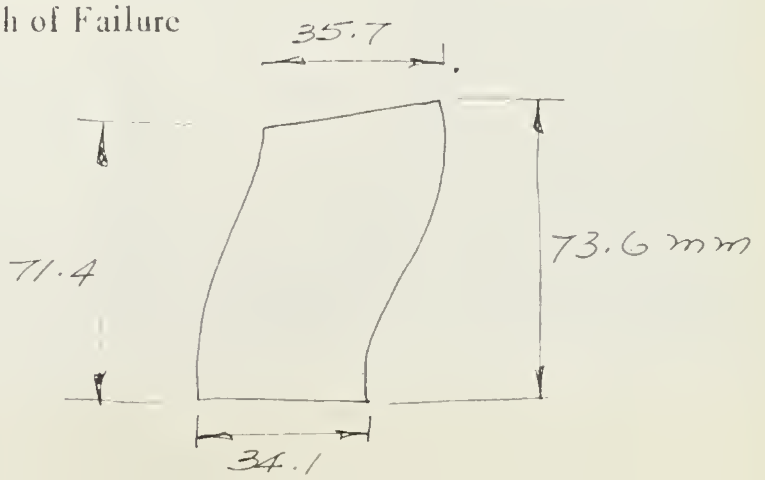
Final 100 (Assumed)

Ratio – Initial 1.456

Final 1.24 (based on e = w. Gs)

ressure Reaction 92%

Sketch of Failure



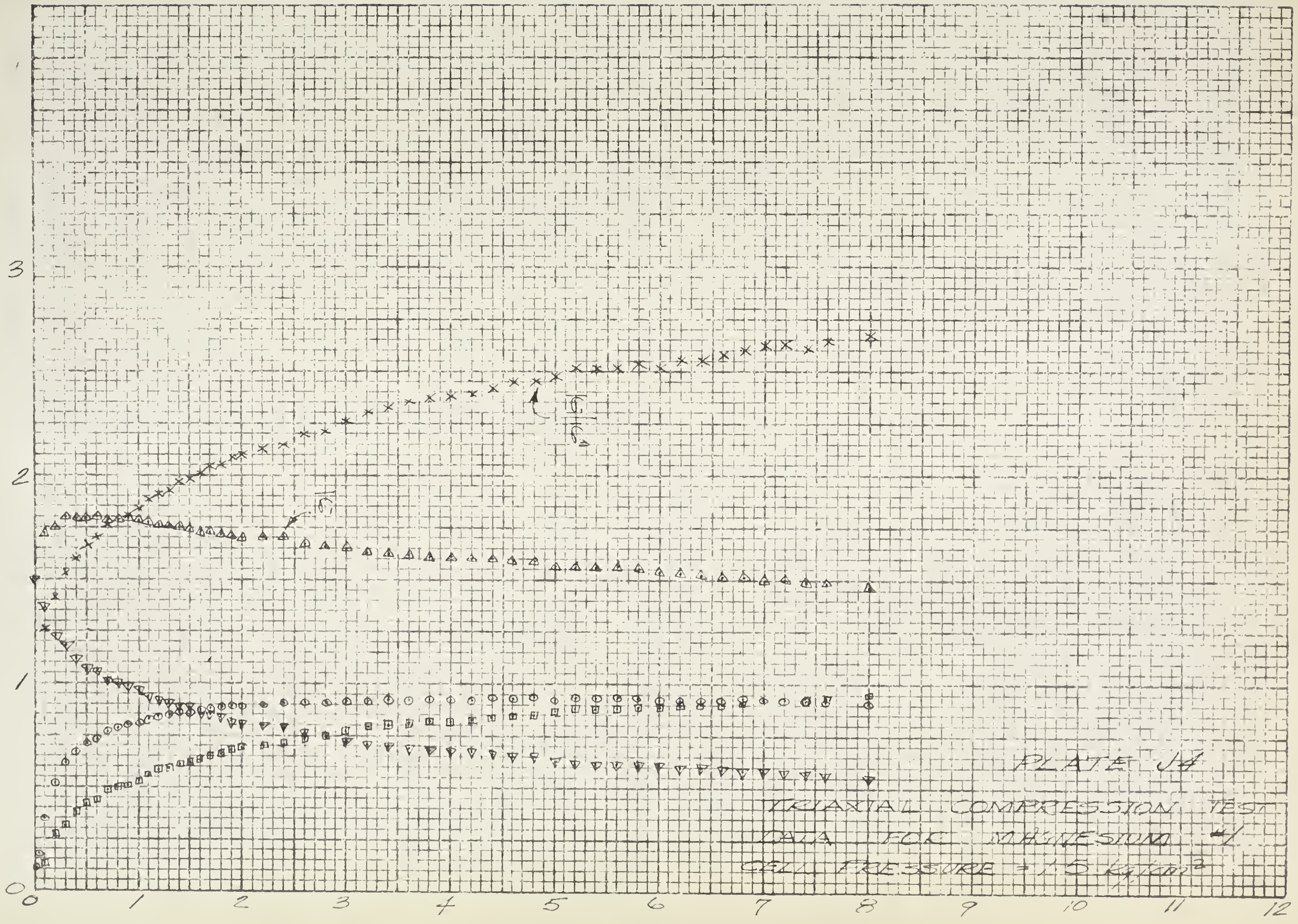


PLATE J4

TRIAXIAL COMPRESSION TEST
DATA FOR MAGNESIUM #1
CELL PRESSURE = 1.5 kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. MAGNESIUM
Depth sample 2
Engineer S.T.
Technician DW
Date of Set-up 19 JUN 61
Test Lateral Pressure 2.75 kg/cm²

ample Description C.E.C 32.6 me/100gm
Adsorbed Cations Ca 9.0, Na 0.6,
K 3.1, Mg 87.3; SC 1.8 me/100gm.

nitial Weight 132.426 gms

ngth, mm	1. 80.44	2. 80.34	Aver: 80.39	Area Top A_T 10.156 sq.mm
am. mm Top	1. 36.10	2. 35.82	Aver: 35.96	Area Centre A_C 9.976 sq.mm
Centre	1. 35.70	2. 35.58	Aver: 35.64	Area Bottom A_B 9.770 sq.mm
Bottom	1. 35.34	2. 35.20	Aver: 35.27	Average X-Sect Area

ginal Volume 80.15 cc

Average X-Sect Area $= \frac{1}{4}(A_T + 2A_C + A_B)$ = 9.97

NSOLIDATION DATA

ate me	Δt min.	Burette Rdg. c.c.	ΔV c.c.
N 1515	0	24.70	
	.1	23.90	
	.25	23.80	
	.5	23.65	
	1	23.40	
	2	22.95	
	4	22.20	
	6	21.50	
	8	20.90	
	~~8~~ 10	20.40	
	15	19.35	
	25	17.95	
	30	17.35	
	45	16.10	
	75	14.35	
	100	13.40	
1925	250	11.20	
2130	375	10.65	
0840	1045	9.95	
1705	1550	9.75	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
21 JUN 0840	2485	9.65	
22 " 1615	4380	9.65	15.05
		K = 1.155	

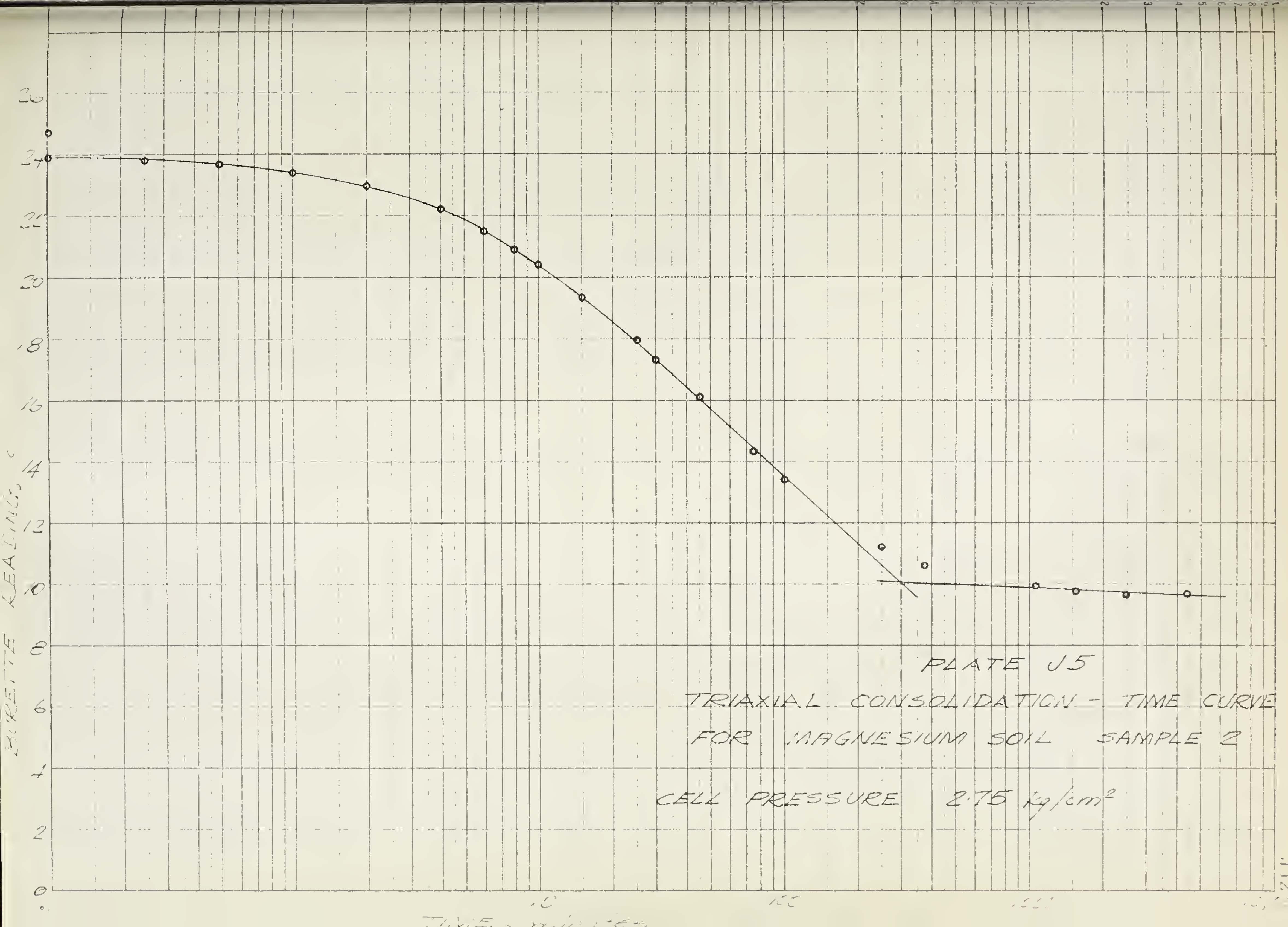


PLATE J5

TRIAXIAL CONSOLIDATION - TIME CURVE
FOR MAGNESIUM SOIL SAMPLE 2

CELL PRESSURE 2.75 kg/cm²

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. MAGNESIUM Gp 1
Depth Sample 2
Engineer ST
Technician DW
Date of Test 23 JUN 61
Test Lateral Pressure σ_3 2.75 kg/cm²
Back Pressure 2.0
Remarks Proving Ring 3-1498
Rate of Strain 552 min/10 mm.

Area Correction Factor 1.155

at Failure
$- \sigma_3) =$ 1.63 $\sigma_1 =$ 4.38
1.71 kg/cm²
2.67
1.04
n 11.20 %

ne n 00	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² Pp	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_p}{\sigma_1 - \sigma_3}$
0	0	9.97	0		0	0					
4	1	✓	20	.0525	.12	.01				.01	
8	3	✓	40	.0511	.24	.05				.04	
7	8	9.98	89	.0490	.50	.17	3.08	2.58	1.19	.1	34.0
28	16	9.99	137	.0475	.75	.32	3.18	2.43	1.31	.2	42.7
37	24	10.00	167	.04685	.90	.43	3.22	2.32	1.39	.3	47.8
3	32	10.01	187	.04650	1.00	.51	3.24	2.24	1.45	.4	54.0
5	48	10.03	214	.04606	1.14	.67	3.22	2.08	1.55	6	58.8
2	56	10.04	225	.04590	1.19	.70	3.24	2.05	1.58	.7	58.8
7	64	10.05	231	.04580	1.22	.76	3.21	1.99	1.61	.8	62.3
3	72.5	10.06	239	.04572	1.25	.80	3.20	1.95	1.64	.9	64.0
8	80.5	10.07	244	.04565	1.28	.84	3.19	1.91	1.67	1.00	65.6
5	88.5	10.08	249	.04560	1.30	.86	3.19	1.89	1.69	1.1	66.1
7	96.5	10.09	253	.04555	1.32	.92	3.15	1.83	72	1.2	69.9
4	104.5	10.10	256	.04552	1.33	.93	3.15	1.82	73	1.3	69.9
9	112.5	10.11	260	.04547	1.35	.99	3.11	1.76	77	1.4	73.3
4	120.5	10.12	263	.04542	1.36	1.02	3.09	1.73	1.79	1.5	75.0
9	128.5	10.13	266	.04540	1.38	1.03	3.10	1.72	1.80	1.6	74.6
4	136.5	10.14	268	.04537	1.38	1.04	3.09	1.71	1.81	1.7	75.4
0	144.5	10.15	272	.04535	1.40	1.08	3.07	1.67	1.84	1.8	77.1
25	152.5	10.16	274	.04531	1.41	1.10	3.06	1.65	1.8[illegible]	1.9	78.0
30	161	10.17	277	.04528	1.42	1.13	3.04	1.62	1.8[illegible]	2.0	79.6
40	177	10.19	281	.04525	1.44	1.16	3.03	1.59	1.91	2.2	80.6
50	193	10.21	284	.04521	1.45	1.20	3.00	1.55	1.94	2.4	82.7
60	209	10.23	289	.04515	1.47	1.24	2.98	1.51	1.97	2.60	84.3
70	225	10.26	291	.04514	1.48	1.28	2.95	1.47	2.01	2.8	86.4
9	241	10.27	294	.04511	1.49	1.30	2.94	1.45	2.03	3.0	87.2
0	257	10.29	298	.04507	1.51	1.33	2.93	1.42	2.06	3.2	88.0
0	273	10.31	300	.04505	1.51	1.38	2.88	1.37	2.10	3.4	91.4

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Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. MAGNESIUM

Depth Sample 2

Area Correction Factor 1.155

me in	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg cm^2 P_r	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_f}{\sigma_1-\sigma_3}$
10	289.5	10.335	302	.04502	1.52	1.39	2.88	1.36	2.12	3.6	91.4
19	305.5	10.36	305	.0450	1.53	1.43	2.85	1.32	2.16	3.8	93.4
29	321.5	10.38	306	.04499	1.53	1.43	2.85	1.32	2.16	4.0	93.4
39	337.5	10.41	308	.04497	1.54	1.45	2.84	1.30	2.18	4.2	94.1
50	354	10.43	311	.04495	1.55	1.48	2.82	1.27	2.22	4.4	95.4
60	370	10.45	312	.04494	1.55	1.49	2.81	1.26	2.23	4.6	96.1
69	386	10.47	313	.04492	1.55	1.51	2.79	1.24	2.25	4.8	97.4
79	402	10.49	316	.0449	1.56	1.53	2.78	1.22	2.28	5.0	98.1
90	418	10.51	317	04490	1.56	1.52	2.79	123	2.27	5.2	97.4
00	434	10.54	319	.04485	1.57	1.53	2.79	1.22	2.29	5.4	97.5
10	450	10.56	321	.04485	1.57	1.54	2.78	1.21	2.30	5.6	98.1
20	466.5	10.58	323	.0448	1.58	1.55	2.78	1.20	2.32	5.8	98.1
30	482.5	10.60	325	.04480	1.59	1.56	2.78	1.19	2.34	6.0	98.1
41	502.5	10.64	327	.04480	1.59	1.58	2.76	1.17	2.36	6.25	99.4
54	522.5	10.66	329	.04478	1.60	1.59	2.76	1.16	2.38	6.5	99.4
6	542.5	10.69	331	.04477	1.60	1.59	2.76	1.16	2.38	6.75	99.4
78	562.5	10.72	334	.04475	1.61	1.61	2.75	1.14	2.41	7.	100
90	583	10.74	335	.04473	1.61	1.63	2.73	1.12	2.44	7.25	101.2
2	603	10.77	337	.04470	1.61	1.61	2.75	1.14	2.41	7.5	100
4	623	10.80	338	.04470	1.61	1.62	2.74	1.13	2.42	7.75	100.6
26	643	10.83	339	.04469	1.62	1.63	2.74	1.12	2.45	8.0	100.6
39	663	10.86	341	.04468	1.62	1.63	2.74	1.12	2.45	8.25	100.6
150	683	10.89	343	.04466	1.62	1.64	2.73	1.11	2.46	8.50	101.2
163	703.5	10.92	344	.04465	1.62	1.65	2.72	1.10	2.47	8.75	101.8
75	723.5	10.95	346	.04463	1.63	1.66	2.72	1.09	2.50	9.0	101.8
87	743.5	10.98	347	.04462	1.63	1.67	2.72	1.08	2.52	9.25	102.5
00	764	11.01	348.5	04461	1.63	1.69	2.69	1.06	2.54	9.5	103.7
82	900	11.22	356	.04455	1.63	1.71	2.67	1.04	2.57	11.20	104.9
2	932.5	11.27	356.5	.04455	1.63	1.69	2.69	1.06	2.54	11.6	103.7
2	965	11.32	357	04453	1.62	1.69	2.68	1.06	2.53	12.0	104.5

ure Content – Initial 55.95

Final 39.88

e of Saturation – Initial 98.0

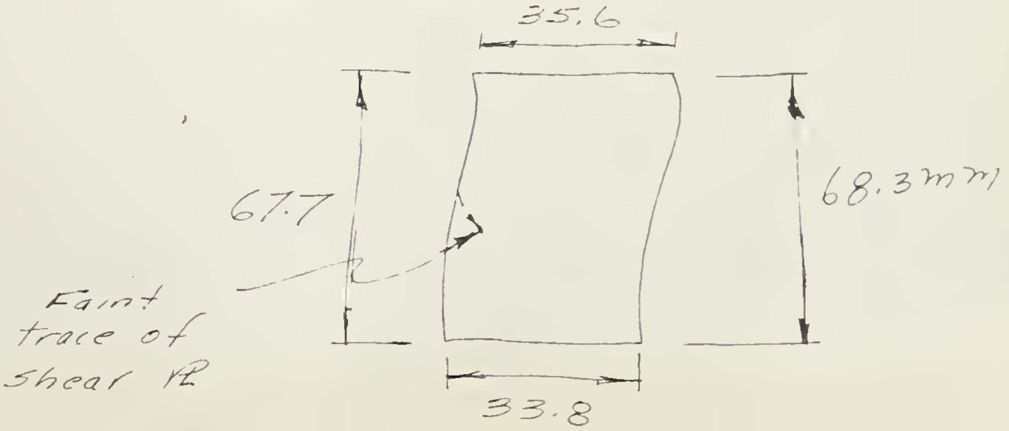
Final 99.9

Ratio – Initial 1.529

Final 1.070

ressure Reaction 83%

Sketch of Failure



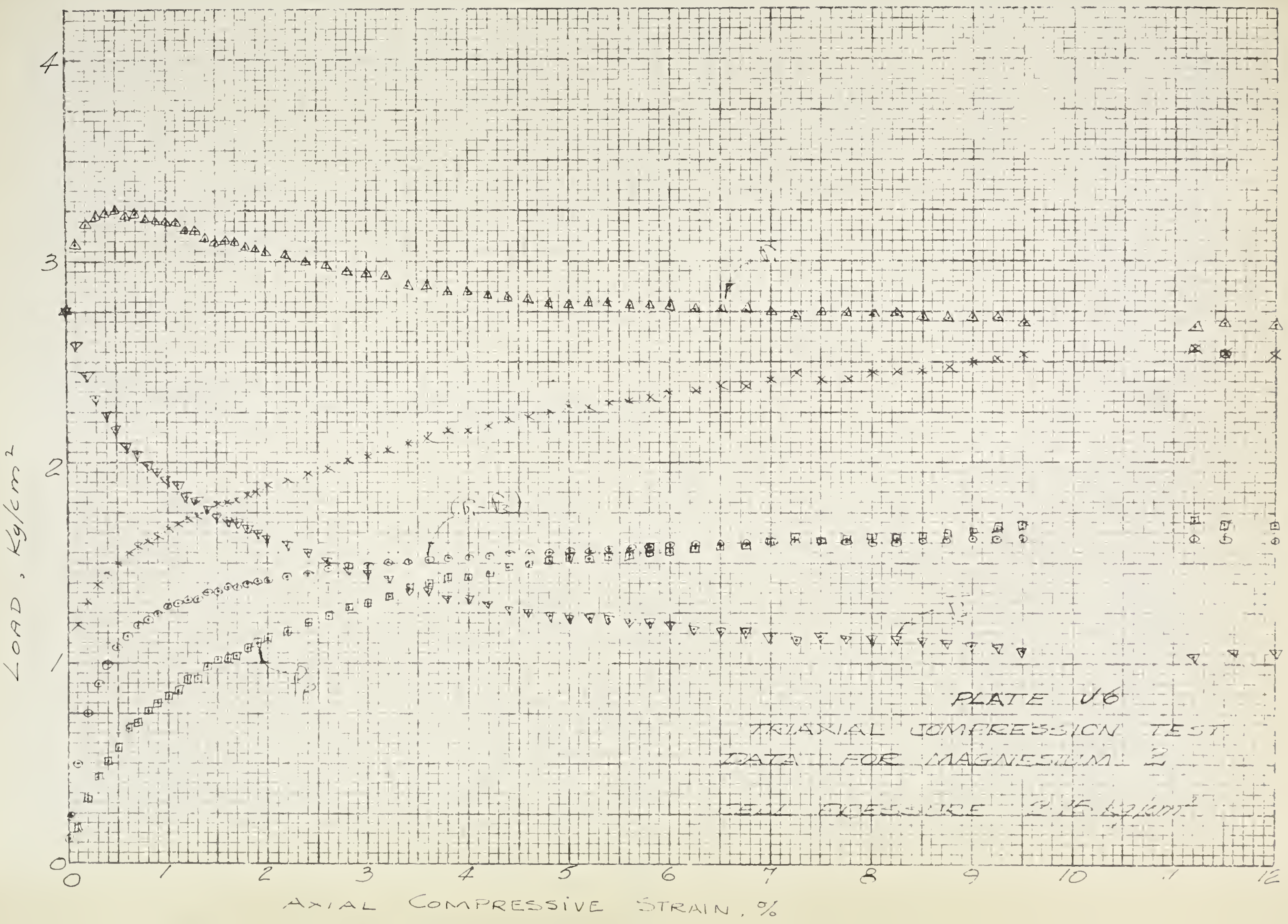


PLATE U6

TRIAXIAL COMPRESSION TEST

DATA FOR MAGNESIUM 2

CELL PRESSURE 2.15 kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. MAGNESIUM GP1

Depth ______ sample 3

Engineer ST

Technician DW

Date of Set-up 19 JUN 61

Test Lateral Pressure 4 kg/cm²

Sample Description CEC 32·6 me/100gm soil Adsorbed Cations Ca 9·0, Na 0·6, K= 3·1, Mg 87·3 SC 1·8 me/100gm soil

Initial Weight 135·868 gms

ength, mm	1. 80.76	2. 80.80	Aver: 80·78
am. mm Top	1. 35.36	2. 35.36	Aver. 35·36
Centre	1. 35.92	2. 35.94	Aver: 35·93
Bottom	1. 36·28	2. 36·18	Aver: 36·23

Area Top A_T 9·820 sq.mm

Area Centre A_C 10·139 sq.mm

Area Bottom A_B 10·309 sq.mm

Average X–Sect Area

$= \frac{1}{4}(A_T + 2A_C + A_B)$

= 10·10

iginal Volume 81·59 cc

NSOLIDATION DATA

ate me	△ t min.	Burette Rdg. c.c.	△ V c.c.
UN 0900	0	25·05	
	·1	24·20	
	·25	23·95	
	·5	23·65	
	1	23·20	
	2	22·50	
	4	21·30	
	6	20·40	
	8	19·60	
	10	18·90	
	15	17·50	
	20	16·30	
	30	14·45	
	45	12·60	
	60	11·40	
	100	9·55	
	180	8·20	
	250	7·80	
	330	7·45	
1705	485	7·40	

CONSOLIDATION DATA – cont'd

Date Time	△ t min.	Burette Rdg. c.c.	△ V c.c.
21 JUN 0840	1420	7·25	
23 ✓ 0840	4300	7·30	
25 ✓ 1520	7580	7·30	17·75
		K=	1·186

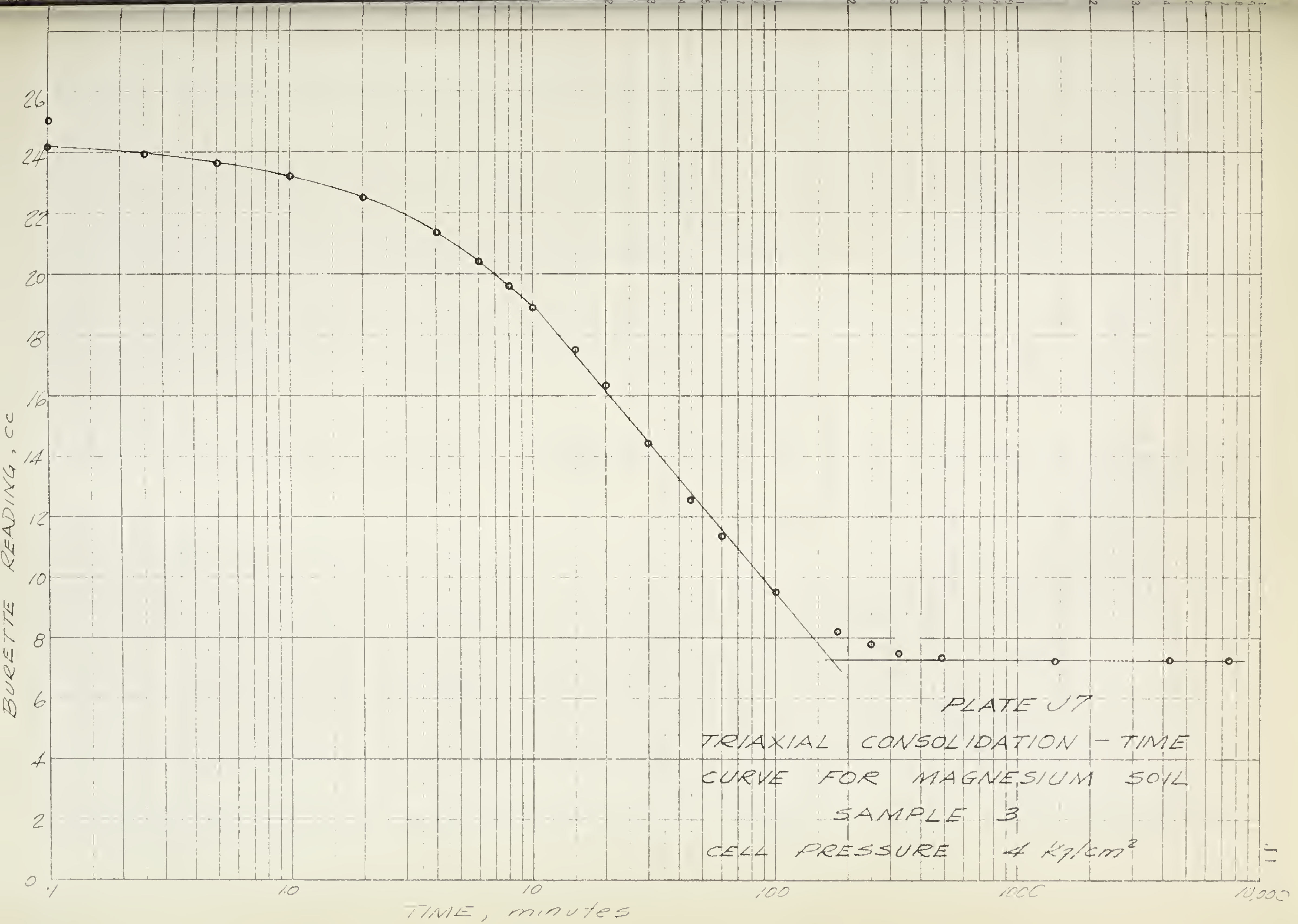


PLATE J7

TRIAXIAL CONSOLIDATION - TIME
CURVE FOR MAGNESIUM SOIL
SAMPLE 3
CELL PRESSURE 4 Kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory
AXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. MAGNESIUM GP1
Depth Sample 3
Engineer ST
Technician DW
Date of Test 4 July 62
Test Lateral Pressure σ_3 4 kg/cm²
Back Pressure 2 kg/cm²
Remarks Proving Ring 3-1498
Rate of Strain 552 min/10mm

at Failure
$- \sigma_3) =$ 2·48 $\sigma_1 =$ 6·48
2.29 kg/cm²
4·19
1.71
in 11.50 %

Area Correction Factor 1.186

me in 850	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_F	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}}{P_p}$ $\sigma_1 - \sigma_3$
0	0	10.10	0		0	+.02					
25	7	10.10	20	.0525	.11	+.06	4.05	3.94	1.03	.09	54.5
29	10	✓	40	.0511	.24	.14	4.10	3.86	1.06	.12	
32	11	10.11	60	.0501	.36	.19	4.17	3.81	1.09	.14	52.7
5	13	10.11	80	.0493	.46	.22	4.24	3.78	1.12	.16	
43	16	10.12	129	.0477	.72	.37	4.35	3.63	1.20	.20	51.3
58	24	10.13	220	.0460	1.19	.66	4.53	3.34	1.36	.30	55.4
1	32.5	10.14	286	.0452	1.51	.85	4.66	3.15	1.48	.40	56.2
2	40.5	10.15	323	.04485	1.69	.98	4.71	3.02	1.56	.50	58.0
9	48.5	10.16	347	.04463	1.81	1.05	4.76	2.95	1.61	.60	58.0
6	56.5	10.17	366	.04447	1.90	1.16	4.74	2.84	1.67	.70	61.1
03	64.5	10.18	380	.04436	1.96	1.20	4.76	2.80	1.70	.80	61.2
09	72.5	10.19	391	.04428	2.02	1.25	4.77	2.75	1.73	.90	61.9
5	80.5	10.20	400	.04422	2.06	1.31	4.75	2.69	1.77	1.00	63.5
21	89.0	10.21	408	.04417	2.09	1.33	4.76	2.67	1.78	1.10	63.6
27	97	10.22	414	.04411	2.12	1.36	4.76	2.64	1.80	1.20	64.1
32	105	10.23	420	.04408	2.15	1.40	4.75	2.60	1.83	1.30	65.1
9	113	10.24	426	.04403	2.17	1.43	4.74	2.57	1.84	1.40	65.8
44	121	10.25	431	.04400	2.19	1.47	4.72	2.53	1.87	1.50	67.1
49	129	10.26	435	.04398	2.21	1.50	4.71	2.50	1.88	1.60	67.8
55	137.5	10.27	439	.04395	2.23	1.53	4.70	2.47	1.90	1.70	68.6
60	145.5	10.28	443	.04393	2.24	1.55	4.69	2.45	1.91	1.80	69.2
5	153.5	10.29	446	.04391	2.26	1.58	4.68	2.42	1.93	1.90	69.9
1	161.5	10.31	450	.04389	2.27	1.59	4.68	2.41	1.94	2.0	70.0
82	177.5	10.33	457	.04385	2.30	1.63	4.67	2.37	1.97	2.2	70.8
2	194	10.35	462	.04382	2.32	1.67	4.65	2.33	2.00	2.4	72.0
01	210	10.37	467	.04380	2.34	1.71	4.63	2.29	2.02	2.6	73.1
1	226	10.39	471.5	.04376	2.36	1.74	4.62	2.26	2.04	2.8	73.7
2	242	10.41	477	.04372	2.38	1.76	4.62	2.24	2.06	3.0	73.9

Sheet 2
J19

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory
RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project
Hole No.
Depth Sample
Area Correction Factor

me in	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm^2 P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\,P_p}{\sigma_1-\sigma_3}$
33	258.5	10.43	481	.04370	2.39	1.81	4.58	2.19	2.09	3.2	75.7
43	275	10.45	4845	.04369	2.40	1.85	4.55	2.15	2.12	3.4	77.1
52	291	10.47	487	.04367	2.41	1.87	4.54	2.13	2.13	3.6	77.6
62	307	10.49	491	.04365	2.42	1.90	4.52	2.10	2.15	3.8	78.5
72	323	10.52	493	.04363	2.42	1.93	4.49	2.07	2.17	4.0	79.8
97	363.5	10.57	500	.04360	2.44	1.99	4.45	2.01	2.21	4.5	81.6
22	404	10.63	507	.04356	2.46	2.03	4.43	1.97	2.25	5.0	82.5
45	444.5	10.69	512	.04354	2.47	2.08	4.39	1.92	2.29	5.5	84.2
71	484.5	10.74	516.5	.04350	2.48	2.11	4.37	1.89	2.31	6.0	85.0
95	525	10.80	521	.04349	2.49	2.14	4.35	1.86	2.34	6.5	85.9
21	565.5	10.86	524.5	.04348	2.49	2.17	4.32	1.83	2.36	7.0	87.1
46	606	10.92	528	.04345	2.49	2.20	4.29	1.80	2.38	7.5	88.4
71	646	10.97	532	.04343	2.50	2.22	4.28	1.78	2.40	8.0	88.8
95	686.5	11.03	536	.04340	2.50	2.25	4.25	1.75	2.43	8.5	90.0
77	824	11.25	545	.04338	2.49	2.27	4.22	1.73	2.44	10.2	91.2
92	848	11.28	547	.04336	2.49	2.28	4.21	1.72	2.45	10.5	91.5
5	868	11.31	548	.04336	2.49	2.29	4.20	1.71	2.46	10.75	92.0
7	888.5	11.35	550	.04335	2.49	2.29	4.20	1.71	2.46	11.0	92.0
29	909	11.38	551	.04334	2.49	2.29	4.20	1.71	2.46	11.25	92.0
40	929	11.41	551	✓	2.48	2.29	4.19	1.71	2.45	11.50	92.3
52	949	11.44	552	.04333	2.48	2.30	4.18	1.70	2.46	11.75	92.7
5	969	11.47	552	✓	2.47	2.32	4.15	1.68	2.47	12.00	93.9

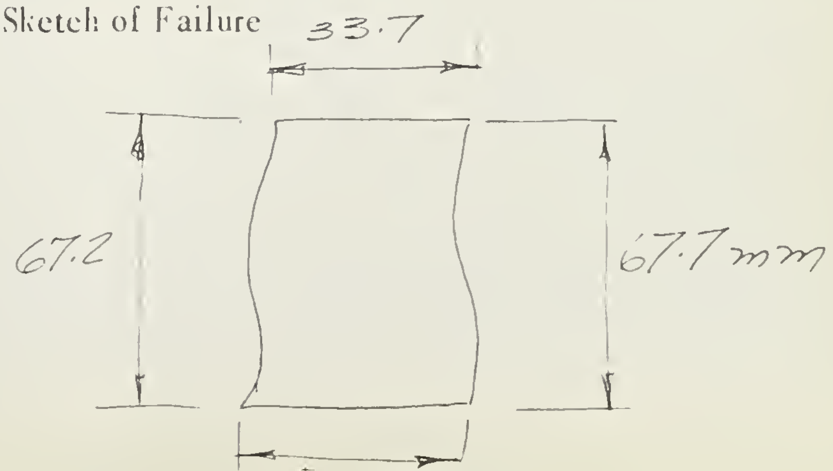
re Content – Initial 53.42
Final 35.11 %

of Saturation – Initial 97.4
Final 98.5

atio – Initial 1.469
Final 0.955

ressure Reaction

Sketch of Failure



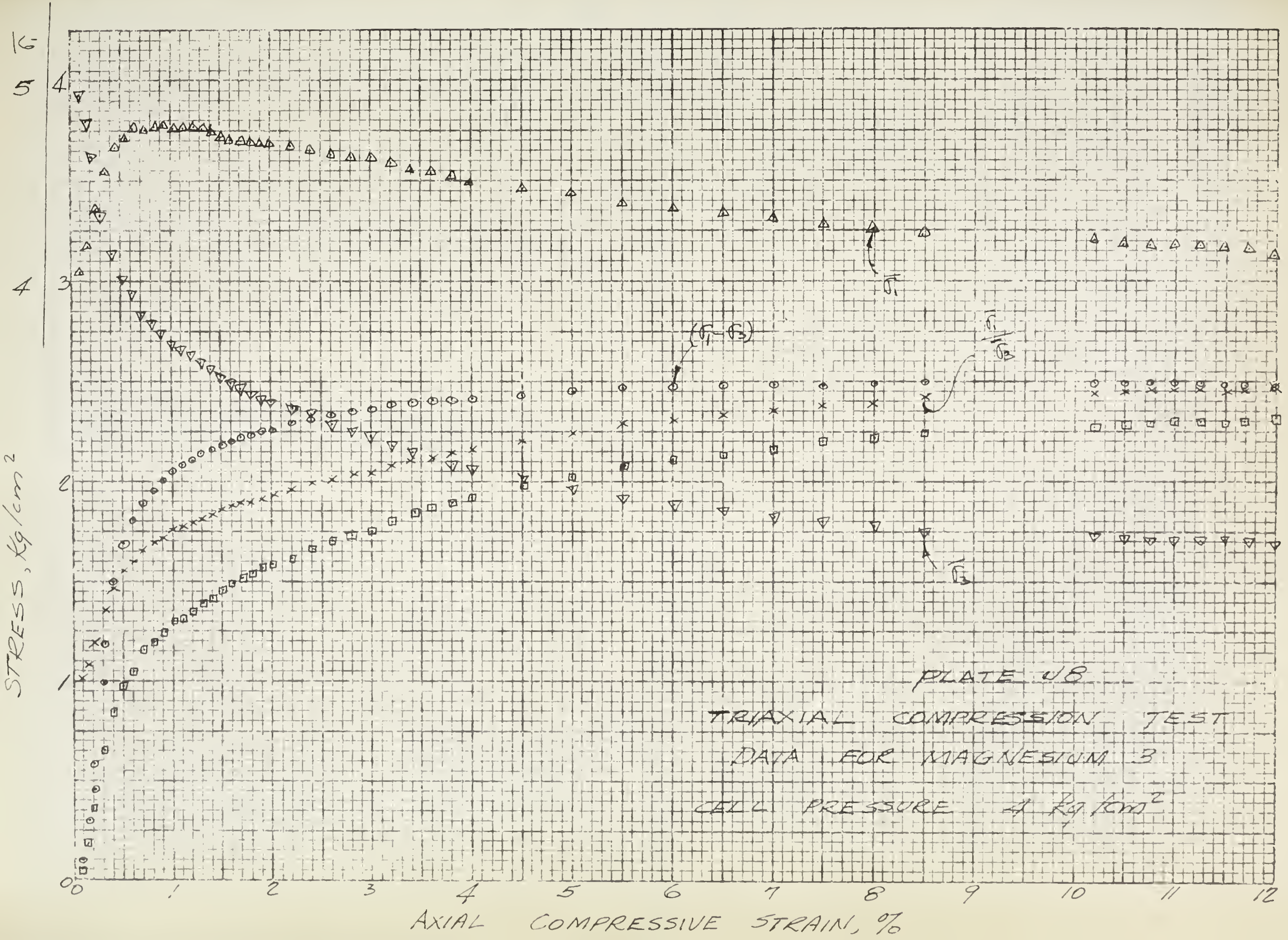


PLATE U8

TRIAXIAL COMPRESSION TEST

DATA FOR MAGNESIUM 3

CELL PRESSURE 4 Kg/cm²

UNIVERSITY OF ALBERT
Department of Civil Engineering
Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Sample Description Cation Exchange Capacity 32.6 m.e./100gm soil. Average Adsorbed Cations: Ca 5.7, Na 0 K 2.1, Mg 92.2% ; Salt Content of pore water 16.5 me/100gm soil

Initial Weight 136.315 gms

Project THESIS
Hole No. MAGNESIUM Gp 2
Depth sample 4
Engineer ST
Technician DW
Date of Set-up 27 July 61
Test Lateral Pressure 5.25 kg/cm²

ength, mm	1. 80.62	2. 80.70	Aver: 80.66	Area Top A_T 10.395 sq.mm
am. mm Top	1. 36.36	2. 36.40	Aver: 36.38	Area Centre A_C 10.128 sq.mm
Centre	1. 35.96	2. 35.86	Aver: 35.91	Area Bottom A_B 9.931 sq.mm
Bottom	1. 35.60	2. 35.52	Aver: 35.56	Average X-Sect Area

iginal Volume 81.87 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

$= 10.15$

NSOLIDATION DATA

ate me	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
ly 0840	0	24.40	
	.1	23.30	
	.25	23.05	
	.5	22.75	
	1	22.20	
	2	21.40	
	3	20.70	
	4	20.05	
	5	19.50	
	6	18.90	
	8	17.90	
	10	17.10	
	15	15.30	
	20	13.80	
	30	11.75	
	40	10.30	
	50	9.15	
	60	8.35	
	100	6.60	
	200	5.33	

CONSOLIDATION DATA – cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
	300	5.00	
1520	400	4.90	
1700	500	4.80	
1900	620	4.80	
2230	830	4.73	
29 July 1030	1550	4.70	
30 ✓ 1410	3210	4.70	

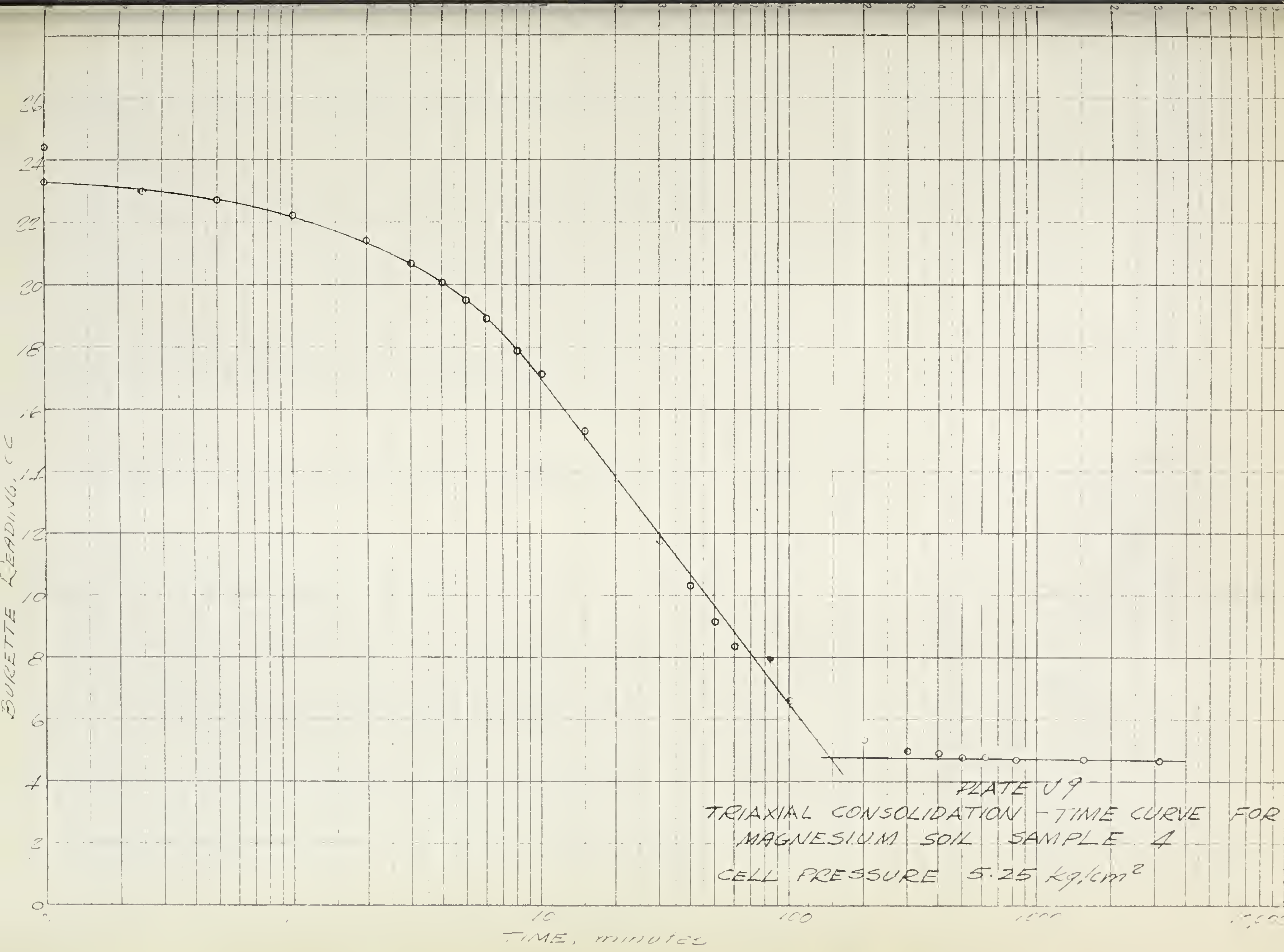


PLATE U9
TRIAXIAL CONSOLIDATION - TIME CURVE FOR
MAGNESIUM SOIL SAMPLE 4
CELL PRESSURE 5.25 kg/cm^2

Sheet 1 J23

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. MAGNESIUM Gp 2

Depth ______ Sample 4

Engineer ST

Technician DW

Date of Test 31 July 61

Test Lateral Pressure σ_3 5.25 kg/cm²

Back Pressure 2 kg/cm²

Remarks Proving Ring 3-1498
Rate of Strain 408 min/10 mm

Area Correction Factor 1.22

a at Failure

$_1 - \sigma_3$) = 3.08 σ_1 = ______

= 2.92 kg/cm²

= 5.41

= 2.33

in 12.0 %

ime min 900	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm² P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\ P_p}{\sigma_1 - \sigma_3}$
0	0	10.15	0		0	+.08					
3	2	10.15	20	.0525	.13	+.09				.025	
5	3	10.16	40	.0511	.25	.12				.037	
10	5	10.16	80	.0493	.47	.20	5.52	5.05	1.09	.062	42.5
16	8	10.16	133	.04765	.76	.31	5.70	4.94	1.15	.1	40.8
29	16	10.17	237	.04576	1.30	.57	5.99	4.69	1.28	.2	43.1
39	24	10.18	308	.04497	1.66	.75	6.16	4.50	1.37	.3	45.1
46	32	10.19	353	.04458	1.88	.90	6.23	4.35	1.43	.4	47.9
52	40	10.20	382	.04436	2.03	.97	6.31	4.28	1.47	.5	47.8
57	48	10.21	405	.04418	2.14	1.12	6.27	4.13	1.52	.6	52.3
62	56	10.22	423	.04405	2.22	1.18	6.29	4.07	1.55	.7	53.1
68	64	10.23	440	.04393	2.30	1.26	6.29	3.99	1.58	.8	54.7
72	72.5	10.24	452	.04388	2.36	1.34	6.27	3.91	1.60	.9	56.8
78	80.5	10.26	465	.04380	2.42	1.41	6.26	3.84	1.63	1.0	58.2
83	88.5	10.27	475	.04374	2.47	1.47	6.25	3.78	1.65	1.1	59.5
87	96.5	10.28	485	.04369	2.52	1.51	6.25	3.74	1.~~20~~67	1.2	59.9
91	104.5	10.285	493	.04365	2.55	1.54	6.26	3.71	1.69	1.3	60.4
94	113	10.30	501	.04360	2.59	1.58	6.26	3.67	1.71	1.4	61.0
99	121	10.31	507	.04355	2.61	1.61	6.25	3.64	1.72	1.5	61.7
103	129	10.32	514	.04352	2.64	1.72	6.17	3.53	1.75	1.6	65.1
112	145	10.34	526	.04346	2.70	1.82	6.13	3.43	1.79	1.8	67.4
116	153	10.35	532	.04343	2.72	1.85	6.12	3.40	1.80	1.9	68.0
121	161	10.36	537	.04340	2.74	1.91	6.08	3.34	1.82	2.0	69.0
128	177.5	10.38	545	.04337	2.78	1.95	6.08	3.30	1.84	2.2	70.1
136	193	10.40	554	.04333	2.82	2.00	6.07	3.25	1.87	2.4	70.9
145	210	10.425	562	.04330	2.85	2.04	6.06	3.21	1.89	2.6	71.6
151	226	10.45	569	.04327	2.87	2.08	6.04	3.17	1.91	2.8	72.5
158	242	10.47	576	.04324	2.90	2.17	5.98	3.08	1.94	3.0	74.8
178	282	10.52	590	.04318	2.95	2.25	5.95	3.00	1.98	3.5	76.3

Sheet 2
J24

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. MAGNESIUM Gp 2

Depth Sample 4

Area Correction Factor 1.22

me in	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg cm² p.	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Corr. Strain %	$\frac{\bar{A}}{P_f}$ $\sigma_1 - \sigma_3$
97	322.5	10.57	601	.04313	2.99	2.37	5.87	2.88	2.04	4.0	79.2
15	363	10.63	611	.04310	3.02	2.46	5.81	2.79	2.08	4.5	81.5
35	403	10.68	618	.04306	3.04	2.52	5.77	2.73	2.11	5.0	82.8
53	443.5	10.74	623	.04303	3.05	2.58	5.71	2.67	2.15	5.5	84.9
72	484	10.80	627	.04302	3.05	2.62	5.68	2.63	2.16	6.0	85.9
89	524	10.86	629	.04301	3.04	2.65	5.64	2.60	2.17	6.5	87.2
06	564	10.91	630	.04301	3.03	2.68	5.60	2.57	2.18	7.0	88.4
25	604.5	10.97	631	.04300	3.02	2.70	5.57	2.55	2.184	7.5	89.4
42	645	11.04	631	.04300	3.00	2.78	5.47	2.47	2.215	8.0	92.7
60	684	11.09	635	"	3.00	2.78	5.47	2.47	2.215	8.5	92.7
79	724.5	11.16	648	.04296	3.04	2.82	5.47	2.43	2.25	9.0	92.8
98	766	11.22	658	.04291	3.07	2.84	5.48	2.41	2.27	9.5	92.5
16	806.5	11.28	665	.04289	3.08	2.86	5.47	2.39	2.29	10.0	92.8
36	847	11.34	670	.04287	3.09	2.88	5.47	2.37	2.30	10.5	93.2
53	887	11.41	675	.04286	3.08	2.89	5.44	2.36	2.313	11.0	93.8
71	927.5	11.47	677	.04285	3.09	2.90	5.44	2.35	2.315	11.5	93.8
89	968	11.525	678	.04285	3.08	2.92	5.41	2.33	2.32	12.0	94.8
05	1251	12.01	481	.04370	2.14	2.98	4.41	2.27	1.94	15.5	
37	1332	12.16	420	.04405	1.86	2.98	4.13	2.26	1.83	16.5	160.2

ure Content – Initial 53.72

Final 33.83

e of Saturation – Initial 97.7

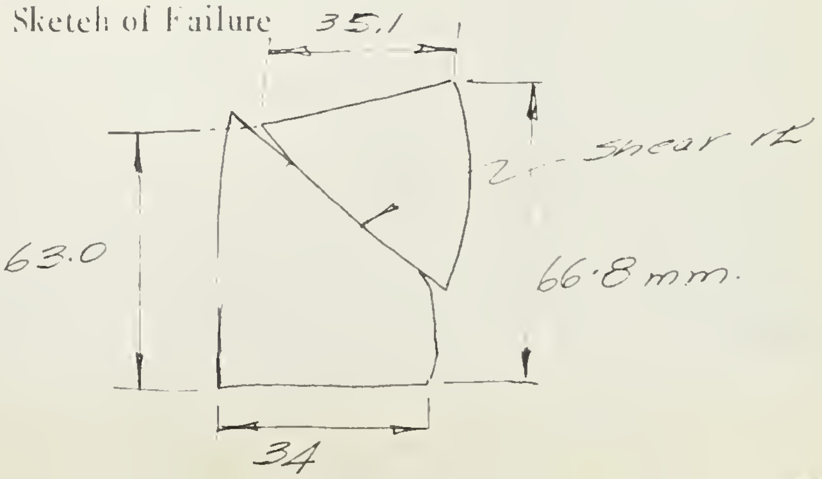
Final 17.4

Ratio – Initial 1.474

Final 0.931

Pressure Reaction 91 %

Sketch of Failure



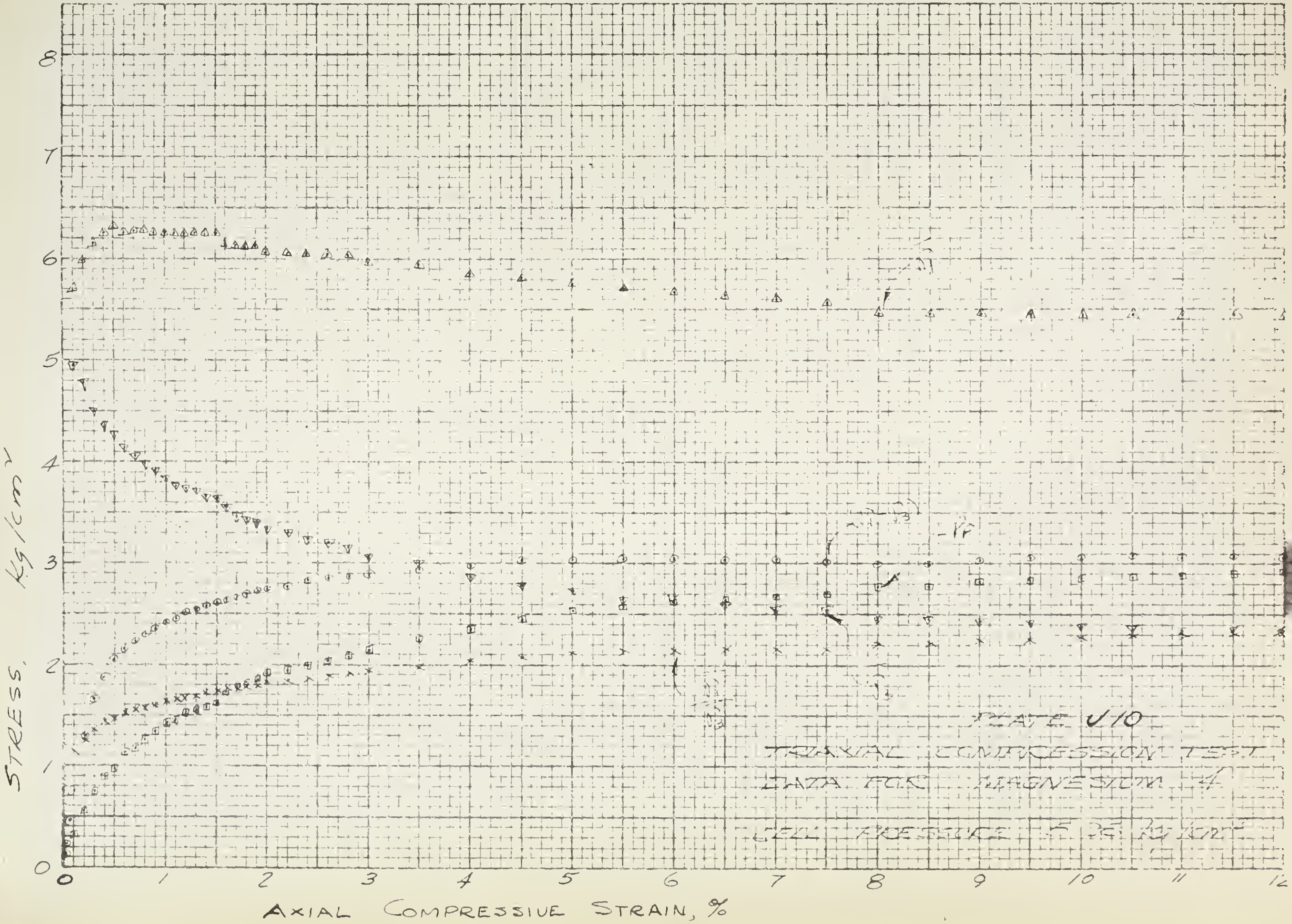


PLATE V10

TRIAXIAL COMPRESSION TEST

DATA FOR MAGNESIUM 4

CELL PRESSURE 5.25 Kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Sample Description CEC 32.6 M.E./100 gm soil
Aver Cations Adsorbed Ca 5.7%
Na 0%, K 2.1%, Mg 92.2%
SC 16.5 m.e /100 gm soil

Initial Weight 129.985 gms

Project THESIS
Hole No. MAGNESIUM (Gp 2)
Depth sample 6
Engineer ST
Technician DW
Date of Set-up 29 July 62
Test Lateral Pressure 7.5 kg/cm²

ngth, mm	1. 78.84	2. 78.86	Aver: 78.85	Area Top	A_T 10.038	sq.mm
am. mm Top	1. 35.78	2. 35.72	Aver. 35.75	Area Centre	A_C 9.904	sq.mm
Centre	1. 35.62	2. 35.40	Aver: 35.51	Area Bottom	A_B 9.776	sq.mm
Bottom	1. 35.34	2. 35.22	Aver: 35.28	Average X-Sect Area		

iginal Volume 78.10 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

= 9.91

ONSOLIDATION DATA

ate ime	Δ t min.	Burette Rdg. c.c.	Δ
July 1030	0	24.90	
	.1	24.20	
	.25	24.05	
	.50	23.60	
	1	23.00	
	2	22.05	
	3	21.30	
	4	20.60	
	6	19.50	
	8	18.50	
	10	17.60	
	15	15.80	
	20	14.35	
	25	13.15	
	30	12.20	
	40	10.60	
	50	9.50	
	60	8.65	
	82	7.40	
	100	6.75	

CONSOLIDATION DATA - cont'd

Date Time	Δ t min.	Burette Rdg. c.c.	Δ V c.c.
	260	5.05	
1930	540	4.75	
30 Jul 1410	1660	4.65	
31 ✓ 0815	2745	4.65	
1 AUG 0840	4210	4.60	
2 ✓ 0800	5655	4.59	
2 ✓ 1420	5990	4.55	20.35
		K = 1.22	

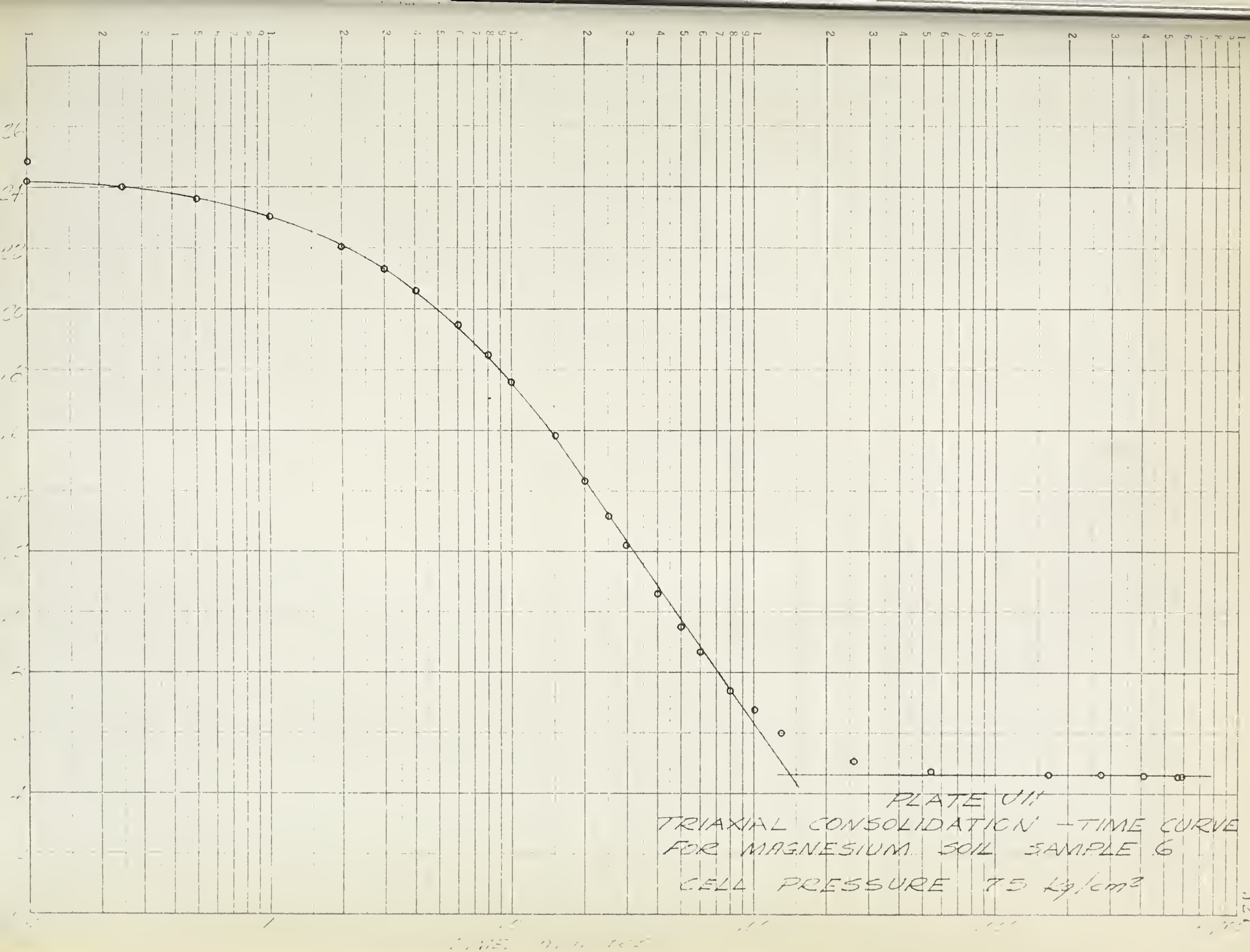


PLATE CII

TRIAXIAL CONSOLIDATION -TIME CURVE
FOR MAGNESIUM SOIL SAMPLE 6

CELL PRESSURE 7.5 Kg/cm²

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. MAGNESIUM Gp 2

Depth

Sample 6

Engineer ST

Technician DW

Date of Test 3 AUG 61

Test Lateral Pressure σ_3 7.5 kg/cm²

Back Pressure 2.0 "

Remarks Proving Ring 3-1498
Rate of Strain 408 min/10 mm

Area Correction Factor 1.22

at Failure

$-\sigma_3) =$ 4.53 $\sigma_1 =$

4.12 kg/cm²

7.91

3.38

12.5 %

ne n /5	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot K}{A_c}$	Pore Press kg/cm² P_F	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}}{\frac{P_p}{\sigma_1-\sigma_3}}$
	0	9.91	0		0						
3	3	✓	20	.0525	.13	+.05				.04	
5	4	✓	40	.0511	.25	.08	7.67	7.42	1.03	.051	32
8	8	9.92	158	.0470	.91	.32	8.09	7.18	1.13	.1	35.2
9	16	9.93	356	.0446	1.95	.84	8.61	6.66	1.29	.2	43.1
3	24	9.94	454	.04386	2.44	1.15	8.79	6.35	1.38	.3	47.1
1	31.5	9.95	510	.04354	2.72	1.40	8.82	6.10	1.45	.4	51.5
8	39.5	9.96	552	.04334	2.93	1.53	8.90	5.97	1.49	.5	52.2
5	47.5	9.97	584	.04320	3.09	1.69	8.90	5.81	1.53	.6	54.7
1	55	9.98	610	.04310	3.21	1.79	8.92	5.71	1.56	.7	55.7
6	63	9.99	630	.04302	3.31	1.90	8.91	5.60	1.59	.8	57.4
1	71	10.00	650	.04295	3.41	2.00	8.91	5.50	1.62	.9	58.6
6	79	10.01	665	.04290	3.48	2.09	8.89	5.41	1.64	1.0	60.0
0	86.5	10.02	678	.04284	3.54	2.16	8.88	5.34	1.66	1.1	61.0
6	94.5	10.03	693	.04280	3.61	2.25	8.86	5.25	1.69	1.2	62.3
2	102.5	10.04	702	.04277	3.65	2.34	8.81	5.16	1.71	1.3	64.1
5	110.5	10.05	712	.04272	3.69	2.39	8.80	5.11	1.72	1.4	64.8
8	118.5	10.06	719	.04269	3.72	2.43	8.79	5.07	1.73	1.5	65.3
2	126	10.07	728	.04265	3.76	2.50	8.76	5.00	1.75	1.6	66.4
6	134	10.08	738	.04262	3.81	2.56	8.75	4.99	1.75	1.7	67.2
30	142	10.09	745	.04260	3.84	2.60	8.74	4.90	1.78	1.8	67.7
5	150	10.10	752	.04258	3.87	2.65	8.72	4.85	1.80	1.9	68.4
9	157.5	10.11	759	.04254	3.90	2.74	8.66	4.76	1.82	2.0	70.3
7	173.5	10.13	771	.04250	3.95	2.80	8.65	4.70	1.84	2.2	70.9
5	189	10.15	783	.04247	4.00	2.87	8.63	4.63	1.86	2.4	71.7
3	205	10.17	794	.04242	4.04	2.96	8.58	4.54	1.89	2.6	73.3
1	221	10.195	804	.04240	4.08	3.02	8.56	4.48	1.91	2.8	74.0
8	236.5	10.215	812	.04236	4.11	3.08	8.53	4.42	1.93	3.0	74.9
7	276	10.27	834	.04228	4.19	3.23	8.46	4.27	1.98	3.5	77.1

UNIVERSITY OF ALBERTA

Department of Civil Engineering
Soil Mechanics Laboratory
RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. MAGNESIUM Gp 2
Depth ______ Sample 6
Area Correction Factor 1.22

me in	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm² P_i	Effective Stress		Stress Ratio	Axial Comp. Strain %	$\frac{\bar{A}\ P_p}{\sigma_1 - \sigma_3}$
							$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor	$\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$		
16	315.5	10.32	852	.04222	4.25	3.39	8.36	4.11	2.03	4.0	79.8
35	355	10.375	867	.04217	4.30	3.46	8.34	4.04	2.06	4.5	80.5
55	394	10.43	883	.04210	4.35	3.53	8.32	3.97	2.10	5.0	81.1
73	433.5	10.485	897	.04206	4.39	3.64	8.25	3.86	2.14	5.5	82.9
93	473	10.54	908	.04202	4.42	3.70	8.22	3.80	2.16	6.0	83.7
11	512.5	10.60	920	.04198	4.45	3.77	8.18	3.73	2.19	6.5	84.7
29	552	10.65	930	.04194	4.47	3.81	8.16	3.69	2.21	7.0	85.2
47	591.5	10.715	939	.04191	4.48	3.87	8.11	3.63	2.23	7.5	86.4
65	631	10.77	949	.04188	4.50	3.90	8.10	3.60	2.25	8.0	86.7
85	670	10.83	956	.04186	4.51	3.92	8.09	3.58	2.26	8.5	86.9
02	709.5	10.89	963	.04184	4.51	3.96	8.05	3.54	2.27	9.0	87.8
20	749	10.95	971	.04181	4.52	4.00	8.02	3.50	2.29	9.5	88.5
38	788.5	11.01	979	.04179	4.53	4.04	7.99	3.46	2.31	10.0	89.2
56	828	11.07	986	.04177	4.54	4.06	7.98	3.44	2.32	10.5	89.4
74	867	11.135	992	.04175	4.54	4.08	7.96	3.42	2.327	11.0	89.9
92	907	11.20	997	.04173	4.53	4.09	7.94	3.41	2.328	11.5	90.3
10	946	11.26	1003	.04170	4.53	4.10	7.93	3.40	2.332	12.0	90.5
29	985.5	11.325	1009	.04170	4.53	4.12	7.91	3.38	2.340	12.5	90.9 ←
46	1025	11.39	1011	.04169	4.51	4.12	7.89	3.38	2.334	13.0	91.4
62	1064	11.46	1013	.04168	4.50	4.12	7.88	3.38	2.331	13.5	91.6
80	1104	11.52	1013	.04168	4.47	4.12	7.85	3.38	2.322	14.0	92.2
15	1183	11.66	1009	.04170	4.40	4.17	7.73	3.33	2.321	15.0	94.8

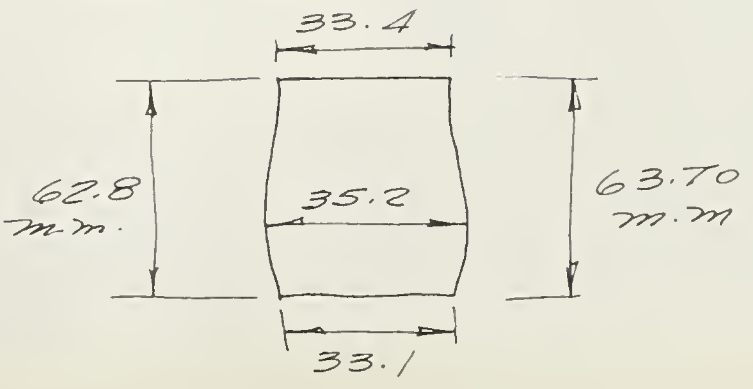
ure Content – Initial 56.89
Final 32.49

e of Saturation – Initial 99.9
Final 101.4 (?)

Ratio – Initial 1.527
Final 0.859

Pressure Reaction 85%

Sketch of Failure



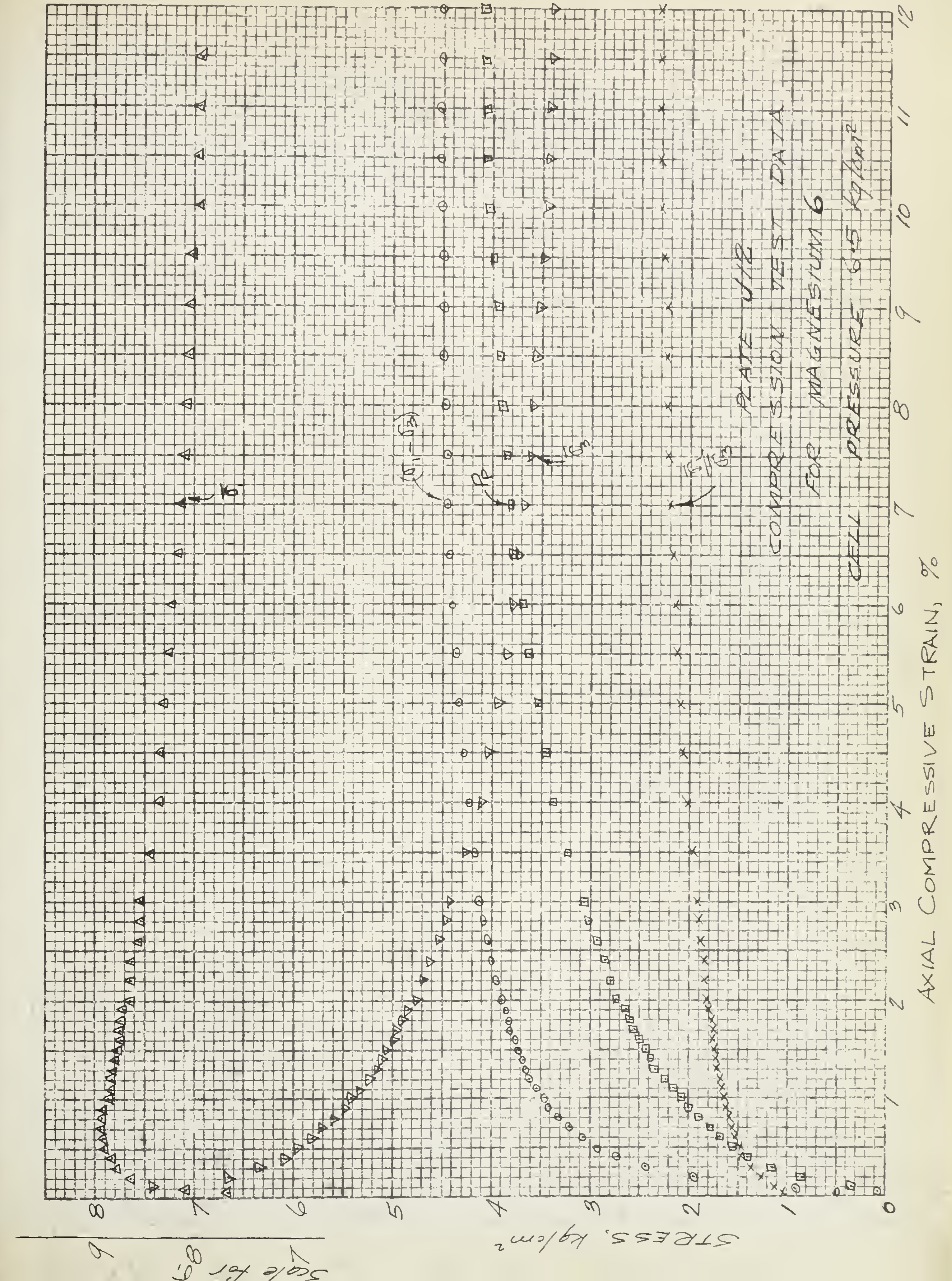


PLATE U18
COMPRESSION TEST DATA
FOR MAGNESIUM 6
CELL PRESSURE 6.5 kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Sample Description CEC 32.6 m.e/100 gm Soil
Aver. Cations adsorbed: Ca 5.7%
Na 0%, K 2.1%, Mg 92.2%
S.C. 16.5 me/100 gm soil

Initial Weight 122.061 gms

Project THESIS
Hole No. MAGNESIUM Gp2
Depth sample 5
Engineer ST
Technician DW
Date of Set-up 28 July 61
Test Lateral Pressure 6.5 kg/cm^2

ngth, mm	1. 73.74	2. 73.68	Aver: 73.71	Area Top	A_T 9.737 sq.mm
am. mm Top	1. 35.22	2. 35.20	Aver: 35.21	Area Centre	A_C 9.904 sq.mm
Centre	1. 35.54	2. 35.48	Aver: 35.51	Area Bottom	A_B 10.021 sq.mm
Bottom	1. 35.76	2. 35.68	Aver: 35.72	Average X-Sect Area	

ginal Volume 72.91 cc

$= \frac{1}{4}(A_T + 2A_C + A_B)$

= 9.89

NSOLIDATION DATA

ate me	Δt min.	Burette Rdg. c.c.	ΔV c.c.
ul 1515	0	25.00	
	.1	24.45	
	.25	24.20	
	.5	23.80	
	1	23.20	
	2	22.35	
	3	21.60	
	5	20.40	
	6	19.90	
	8	19.00	
	10	18.20	
	15	16.60	
	20	15.30	
	25	14.25	
	30	13.40	
	65	9.90	
	100	8.55	
	120	8.06	
	230	7.05	
2235	440		

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
29 Jul 1030	1155	6.61	
29 ✓ 1930	1695	6.60	
30 ✓ 1410	2815	6.62	
31 ✓ 0815	3900	6.60	
31 2000	4605	6.60	18.40
		K= 1.194	

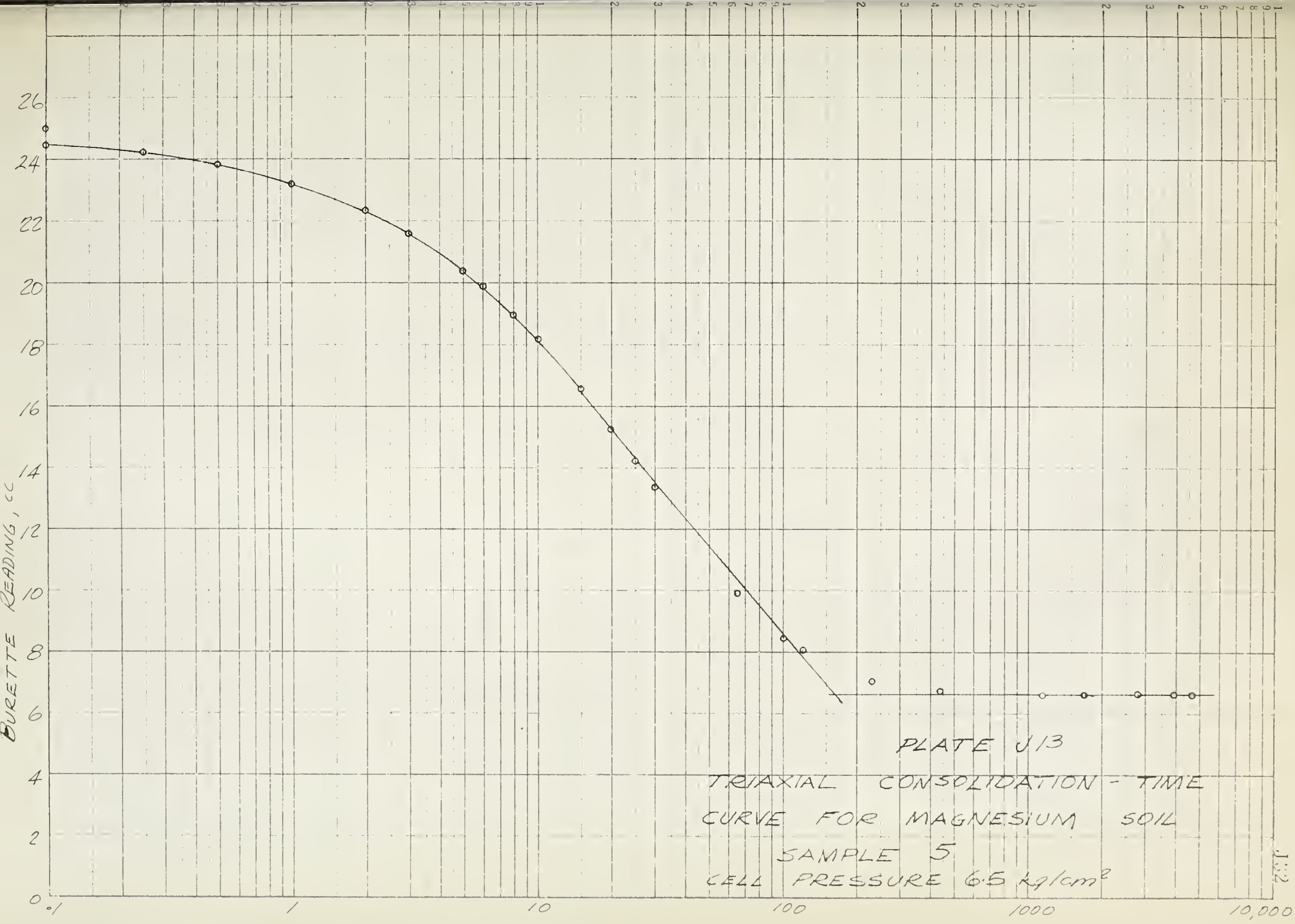


PLATE J13

TRIAXIAL CONSOLIDATION - TIME
CURVE FOR MAGNESIUM SOIL
SAMPLE 5
CELL PRESSURE 6·5 kg/cm²

Sheet 1 J33

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

IAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. MAGNESIUM

Depth Sample 5

Engineer ST

Technician DW

Date of Test 1 AUG 61

Test Lateral Pressure σ_3 6.505

Back Pressure 2

Remarks Proving Ring 3-1498
Rate of Strain 408 min/10mm

Area Correction Factor 1.194

at Failure

$- \sigma_3) =$ 3.75 $\sigma_1 =$ 10.25

3.64 Kg/cm²

6.60

2.86

13.5 %

e	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm² P_p	Effective Stress $\overline{\sigma_1}$ Major	Effective Stress $\overline{\sigma_3}$ Minor	Stress Ratio $\frac{\overline{\sigma_1}}{\overline{\sigma_3}}$	Axial Comp. Strain %	$\frac{\overline{A}}{P_p}$ $\sigma_1-\sigma_3$
3	2	9.89	20	.0525	.13	+.1				.027	
4	6	9.90	115	.04813	.67	+.26				.081	
9	8	9.91	168	.04684	.95	.36	7.14	6.19	1.15	.11	37.9
3	15	9.91	298	.04506	1.62	.74	7.43	5.81	1.28	.2	45.7
3	22	9.92	368	.04446	1.97	.96	7.56	5.59	1.35	.3	48.7
1	29.5	9.925	421	.04407	2.23	1.18	7.60	5.37	1.42	.4	52.9
7	37	9.935	456	.04386	2.40	1.34	7.61	5.21	1.46	.5	55.8
2	44	9.945	481	.04370	2.52	1.46	7.61	5.09	1.50	.6	57.9
8	51.5	9.955	503	.04359	2.63	1.57	7.61	4.98	1.53	.7	59.7
3	59	9.965	522	.04350	2.72	1.69	7.58	4.86	1.56	.8	62.1
	66.5	9.97	538	.04340	2.80	1.75	7.60	4.80	1.58	.9	62.5
	73.5	9.98	552	.04334	2.86	1.83	7.58	4.72	1.61	1.0	64.0
7	81	9.99	563	.04330	2.91	1.91	7.55	4.64	1.63	1.1	65.6
1	88.5	10.00	574	.04325	2.96	1.97	7.54	4.58	1.65	1.2	66.6
5	96	10.01	584	.04320	3.01	2.02	7.54	4.53	1.66	1.3	67.1
	103	10.02	592	.04317	3.05	2.07	7.53	4.48	1.68	1.4	67.8
2	110.5	10.03	600	.04314	3.08	2.15	7.48	4.40	1.70	1.5	69.8
7	118	10.04	608	.04310	3.12	2.20	7.47	4.35	1.72	1.6	70.5
	125.5	10.05	615	.04308	3.15	2.26	7.44	4.29	1.73	1.7	71.7
	132.5	10.06	621	.04306	3.17	2.32	7.40	4.23	1.75	1.8	73.2
	140	10.065	629	.04302	3.21	2.38	7.38	4.17	1.77	1.9	74.1
3	147.5	10.075	636	.04300	3.24	2.42	7.37	4.13	1.78	2.0	74.6
2	162	10.095	647	.04296	3.29	2.50	7.34	4.05	1.81	2.2	76.0
8	177	10.115	656	.04293	3.32	2.58	7.29	3.97	1.84	2.4	77.7
6	191.5	10.13	665	.04290	3.36	2.66	7.25	3.89	1.86	2.6	79.1
4	206.5	10.15	674	.04286	3.40	2.72	7.23	3.83	1.89	2.8	80.0
0	221	10.17	682	.04283	3.43	2.78	7.20	3.77	1.91	3.0	81.0
8	258	10.22	701	.04276	3.50	2.88	7.17	3.67	1.95	3.5	82.3
5	295	10.27	715	.04270	3.55	2.98	7.12	3.57	1.99	4.0	83.9

Sheet 2
J34

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. MAGNESIUM

Depth Sample 5

Area Correction Factor 1.194

ne n	Strain Dial Div.	A_c cm^2	No. of Stress Dial Div.	Proving Ring Const δ, kg Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg cm^2 P_i	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\frac{\bar{A}\,P_F}{\sigma_1-\sigma_3}$
15	331.5	10.31	731	.04264	3.61	3.12	7.04	3.43	2.05	4.5	86.4
30	368.5	10.35	740	.04262	3.64	3.14	7.05	3.41	2.07	5.0	86.3
49	405.5	10.41	751	.04258	3.67	3.25	6.97	3.30	2.11	5.5	88.6
5	442	10.46	760	.04255	3.69	3.30	6.94	3.25	2.14	6.0	89.4
82	479	10.515	769	.04251	3.71	3.34	6.92	3.21	2.16	6.5	90.0
99	516	10.57	777	.04249	3.73	3.40	6.88	3.15	2.18	7.0	91.1
16	553	10.62	785	.04246	3.75	3.45	6.85	3.10	2.21	7.5	92.0
33	590	10.67	793	.04243	3.77	3.48	6.84	3.07	2.23	8.0	92.3
7	663.5	10.78	806	.04238	3.78	3.55	6.78	3.00	2.26	9.0	93.9
85	700	10.83	811	.04236	3.79	3.56	6.78	2.99	2.27	~~9~~.5	93.9
1	737	10.89	818	.04233	3.80	3.59	6.76	2.96	2.28	10.0	94.5
9	774	10.94	824	.04231	3.80	3.60	6.75	2.95	2.29	10.5	94.7
35	811	10.99	830	.04230	3.81	3.63	6.73	2.92	2.30	11.0	95.3
51	847.5	11.06	834	.04228	3.81	3.63	6.73	2.92	2.30	11.5	95.3
8	884.5	11.11	838	.04227	3.81	3.64	6.72	2.91	2.31	12.0	95.5
5	921.5	11.30	842	.04226	3.76 ★	3.66	6.65	2.89	2.301	12.5	96.3
1	958	11.37	846	.04224	3.75	3.67	6.63	2.88	2.302	13.0	96.6
8	995	11.43	850	.04223	3.75	3.69	6.61	2.86	2.311	13.5	97.1 ◄
5	1032	11.50	851	.04223	3.73	3.68	6.60	2.87	2.300	14.0	98.7
1	1069	11.57	852	✓	3.71	3.71	6.55	2.84	2.306	14.5	100
7	1105	11.64	853	.04223	3.70	3.70	6.55	2.85	2.298	15.0	100
0	1179	11.775	853	✓	3.65	3.71	6.49	2.84	2.285	16.0	101.6
2	1253	11.915	842	.04226	3.57	3.73	6.39	2.82	2.266	17.0	104.5

★ Note: Areas prior to this in error by a factor of 1.02 (they are too large) a/c graph. Subsequent areas calculated. Checked back for max prin stress ratio

re Content – Initial 54.39%

Final 34.02

e of Saturation – Initial 99.1

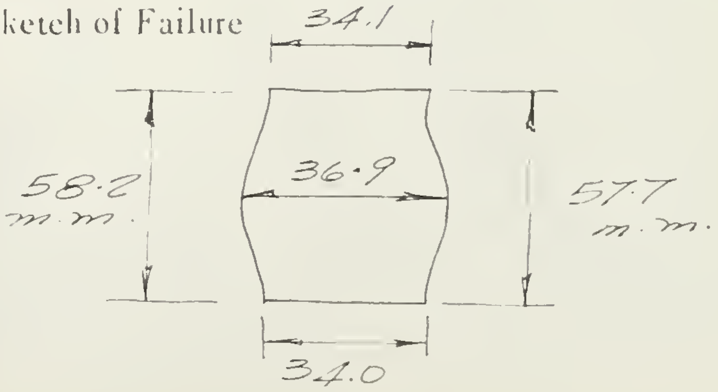
Final 98.8

atio – Initial 1.472

Final 0.923

ressure Reaction 80%

Sketch of Failure



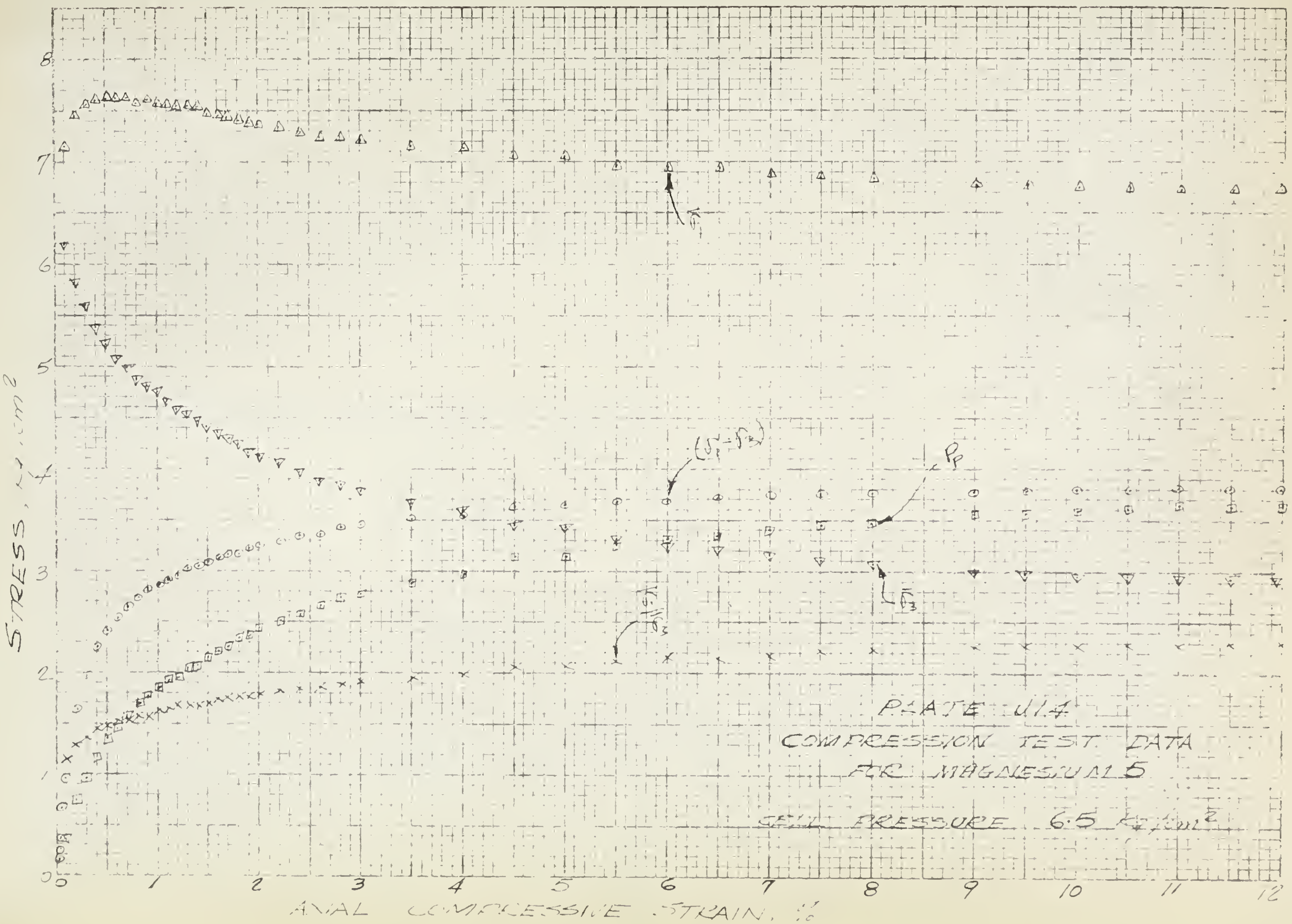


PLATE U14
COMPRESSION TEST DATA
FOR MAGNESIUM 5

CELL PRESSURE 6.5 kg/cm²

UNIVERSITY OF ALBERTA
Department of Civil Engineering
Soil Mechanics Laboratory

TRIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. MAGNESIUM GP 2
Depth sample 7
Engineer ST
Technician
Date of Set–up 9 Feb 62
Test Lateral Pressure 2.75 kg/cm²

Sample Description Samples 4-5-6 crushed & remoulded to form this sample. CEC 32.6 me/100gm Cations: Ca 5.7%, Na 0%, K 2.1% Mg 92.2, S.C 16.5 me/100gm Soil
Initial Weight 134.852 gms

ength, mm	1. ~~73.74~~ 80.64	2. 80.66	Aver: 80.65	Area Top A_T 9.820 sq.mm
iam. mm Top	1. 35.38	2. 35.34	Aver: 35.36	Area Centre A_C 9.965 sq.mm
Centre	1. 35.62	2. 35.62	Aver: 35.62	Area Bottom A_B 10.060 sq.mm
Bottom	1. 35.80	2. 35.78	Aver: 35.79	Average X–Sect Area

riginal Volume 80.27 cc

Average X–Sect Area $= \frac{1}{4}(A_T + 2A_C + A_B)$
$= 9.95$

ONSOLIDATION DATA

ate ime	Δt min.	Burette Rdg. c.c.	ΔV c.c.
eb 1400	0	24.90	
	.1	24.35	
	.25	24.20	
	.50	24.05	
	.75	23.90	
	1	23.75	
	2	23.35	
	3	23.00	
	4	22.75	
	6	22.20	
	8	21.75	
	10	21.35	
	15	20.45	
	20	19.70	
	30	18.45	
	40	17.50	
	60	16.15	
	100	14.60	
1630	150	13.45	
1730	210	12.83	

CONSOLIDATION DATA – cont'd

Date Time	Δt min.	Burette Rdg. c.c.	ΔV c.c.
9Feb 2010	370	11.93	
2210	490	11.65	
10Feb ~~0740~~ 1000	1200	11.20	
1800	1680	11.10	
11Feb1230	2790	11.05	
12Feb0830	3990	11.02	
14Feb1130	7050	10.75	14.15
		K=	1.145

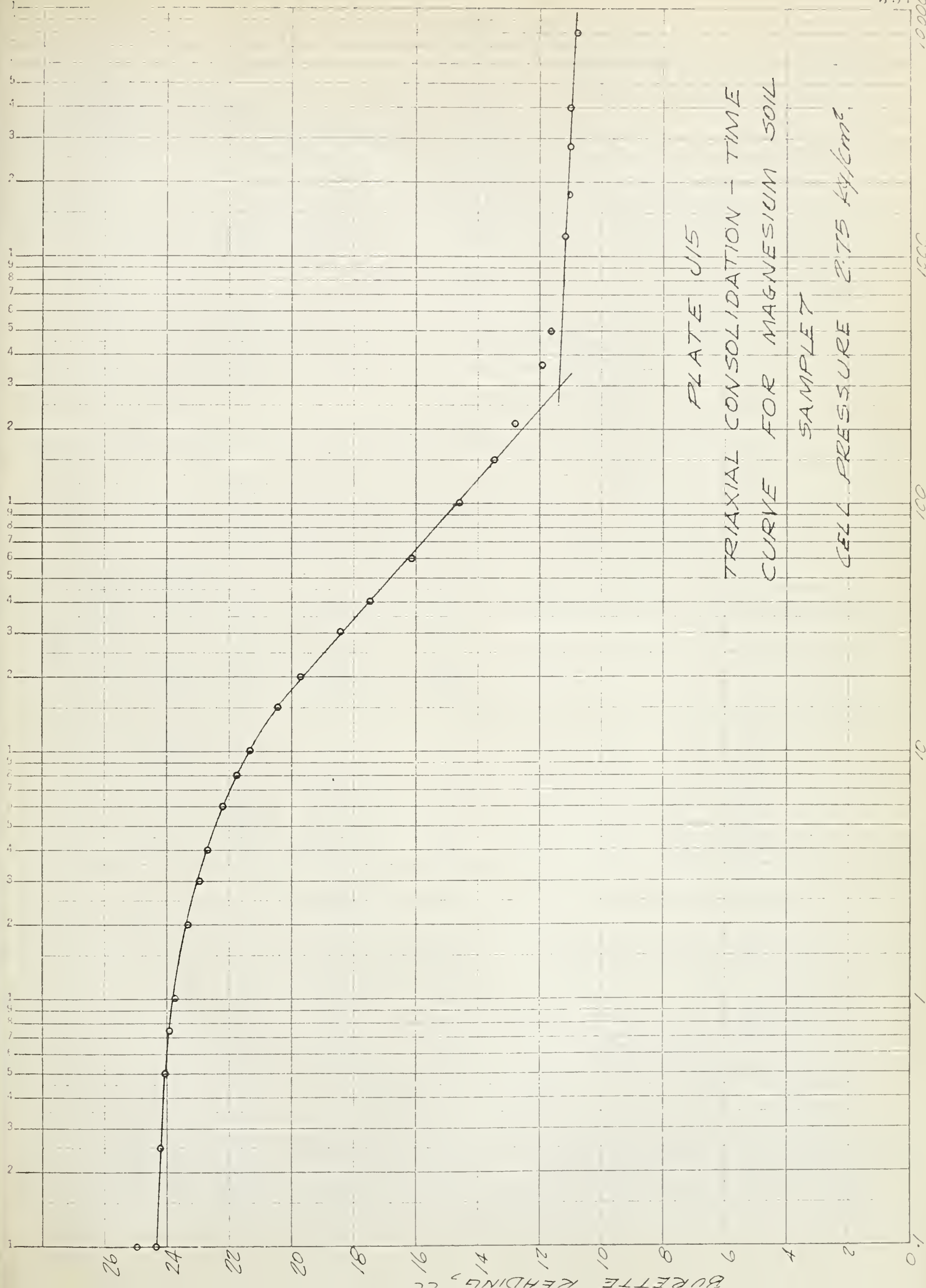


PLATE J/5

TRIAXIAL CONSOLIDATION - TIME
CURVE FOR MAGNESIUM SOIL
SAMPLE 7

CELL PRESSURE 2.75 kg/cm²

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

XIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS
Hole No. MAGNESIUM Gp 2
Depth Sample 7
Engineer ST
Technician W. Semchuk
Date of Test 15 Feb 62
Test Lateral Pressure σ_3 2.75 kg/cm²
Back Pressure 2.0 "
Remarks Remoulded from #4,5 & 6 break-in test on eqpt for W. Semchuk
Area Correction Factor 1.145

at Failure
$- \sigma_3) =$ 1.75 $\sigma_1 =$ 4.50
1.56
2.94
1.19
8.5 %

me n	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg/Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg/cm² P_p	Effective Stress $\bar{\sigma}_1$ Major	Effective Stress $\bar{\sigma}_3$ Minor	Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ $\frac{P_p}{\sigma_1-\sigma_3}$
35											
	0	9.95			0	0					
5	1.5	✓	20	.0525	.12	+.05				.019	
	3.5	✓	40	.0511	.24	+.09				.04	
	5.5	✓	60	.0501	.36	+.13				.07	
0	7	9.955	80	.0493	.45	.20				.09	
4	11	9.96	120	.04798	.66	.27	3.14	2.48	1.266	.14	
9	15.5		160			—					
5	24	9.98	209	.04614	1.11	.50	3.36	2.25	1.493	.3	
2	32	9.99	243	.0457	1.27	.62	3.40	2.13	1.596	.4	48.8
3	40	10.00	260	.04547	1.35	.66	3.44	2.09	1.646	.5	
7	48	10.01	~~45~~ 275	.04530	1.42	.71	3.46	2.04	1.696	.6	50.0
1	56	10.02	286.5	.04519	1.48	.76	3.47	1.99	1.744	.7	51.4
7	72	10.04	302	.04503	1.55	.85	3.45	1.90	1.816	.9	54.8
0	80	10.05	309	.04496	1.58	.91	3.42	1.84		1.0	
3	88	10.06	314	.04492		.91				1.1	
6	96	10.07	319	.04487	1.63	.94	3.44	1.81	1.900	1.2	
9	104	10.08	322	.04485	1.64	.96	3.43			1.3	58.5
1	112	10.09	325	.04483	1.65	1.00				1.4	
4	120	10.10	327	.04480	1.66	1.03	3.38	1.72	1.965	1.5	
7	128	10.11	330	.04477	1.67	1.04	3.38	1.71	1.977	1.6	62.2
0	136	10.12	333	.04475	1.69	1.07	3.37	1.68	2.006	1.7	63.3
3	144	10.13	335			1.07				1.8	
6	152	10.14	337			1.09				1.9	
9	160	10.15	339	.04470	1.71	1.11	3.35	1.64	2.043	2.0	64.9
6	181.5	10.16	343	.04463	1.73	1.15	3.33	1.60	2.081	2.25	66.5
3	201.5	10.21	346	.04465	1.73	1.19	3.29	1.56	2.109	2.50	
0	222	10.23	349	.04461		1.23				2.75	
6	242	10.27	351	.04459	1.74	1.25	3.24	1.50	2.160	3.0	71.8
0	282	10.31	354	.04457	1.75	1.31	3.19	1.44	2.215	3.5	

UNIVERSITY OF ALBERTA

Department of Civil Engineering

Soil Mechanics Laboratory

RIAXIAL COMPRESSION TEST ON COHESIVE SOIL

Project THESIS

Hole No. MAGNESIUM Gpl

Depth Sample 7

Area Correction Factor 1.145

ime in	Strain Dial Div.	A_c cm²	No. of Stress Dial Div.	Proving Ring Const δ, kg Div	$\sigma_1 - \sigma_3 = \frac{k_p \cdot \delta \cdot k}{A_c}$	Pore Press kg cm² P	Effective Stress		Stress Ratio $\frac{\bar{\sigma}_1}{\bar{\sigma}_3}$	Axial Comp. Strain %	$\bar{A}$ p. $\sigma_1 - \sigma$
							$\bar{\sigma}_1$ Major	$\bar{\sigma}_3$ Minor			
34	322	10.36	356	.04455	1.75	1.35	3.15	1.40	2.250	4.0	
47	362	10.42	356.5	.04455	1.75	1.39	3.11	1.36	2.287	4.5	
60	402	10.47	357	.04454	1.74	1.41	3.08	1.34	2.298	5.0	81.0
73	442	10.53	357	.04454	1.73	1.43	3.05	1.32	2.311	5.5	
86	482	10.59	357	.04454	1.72	1.46	3.01	1.29	2.333	6.0	84.9
00	522	10.64	356	.04454	1.71	1.47	2.99	1.28	2.336	6.5	86.0
13	562	10.70	361	.04451	1.72	1.51	2.96	1.24	2.387	7.0	
25	602	10.76	366	04447	1.73	1.53	2.95	1.22	2.418	7.5	
39	642	10.82	370	0.4443	1.74	1.54	2.95	1.21	2.438	8.0	
54	682	10.87	373	.04442	1.75	1.56	2.94	1.19	2.471	8.5	
67	722	10.93	375	.04440	1.74	1.56	2.93	1.19	2.462	9.0	89.7
82	770	10.99	376	.04439	1.74	1.57	2.92	1.18	2.478	9.5	
93	802	11.05	377	.04438	1.73	1.57	2.91	1.18	2.466	10.	90.8
	847	11.12	375	04439	1.72	1.57	2.90	1.18	2.458	10.5	
23	895	11.19	373	04442	1.70	1.58	2.87	1.17	2.453	11.1	
40	927.5	11.24	367	04447	1.66	1.58	2.83	1.17	2.419	11.5	95.2
45	968	11.31	336	.04473	1.52	1.56	2.71	1.19	2.277	12.0	102.6

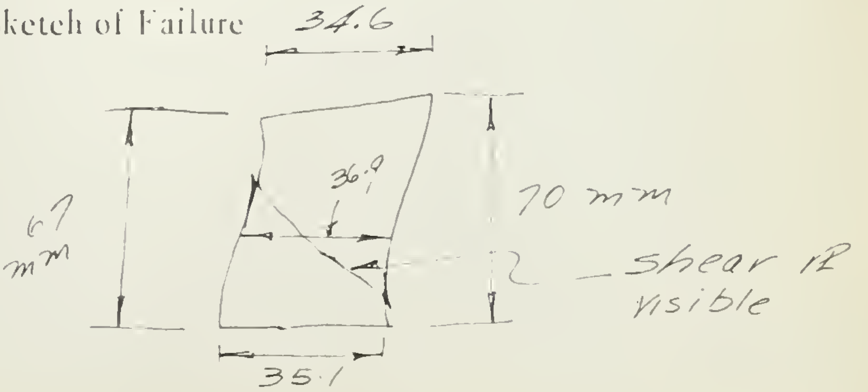
ure Content – Initial 54.2%
Final 39.47

e of Saturation – Initial 99.2
Final 101 (?)

Ratio – Initial 1.46
Final 1.04

ressure Reaction 80%

Sketch of Failure



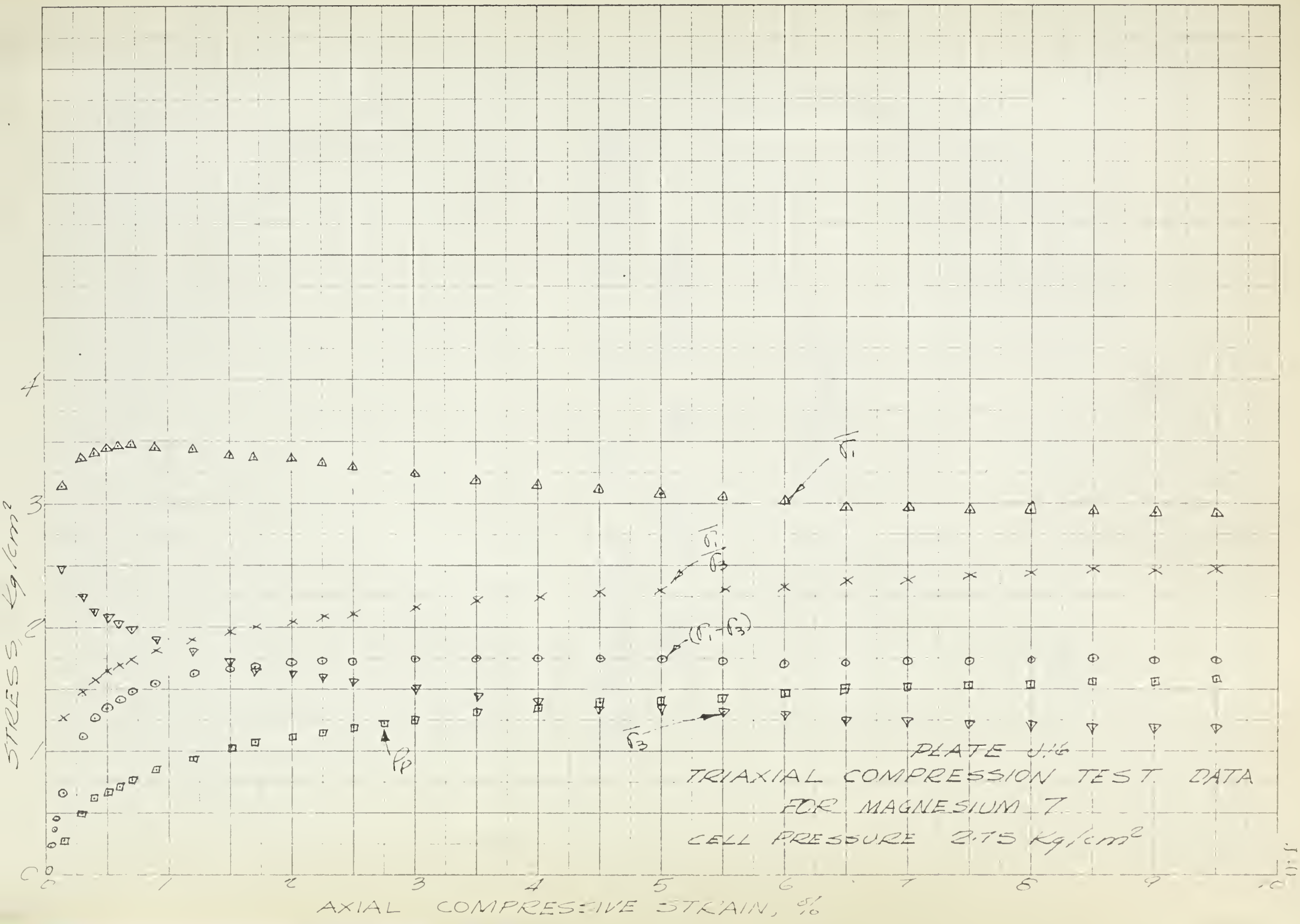


PLATE J.16
TRIAXIAL COMPRESSION TEST DATA
FOR MAGNESIUM 7
CELL PRESSURE 2.75 Kg/cm²

APPENDIX K

RESULTS OF TESTS ON SODIUM-CALCIUM SOIL MODIFICATIONS

APPENDIX K

SODIUM - CALCIUM MODIFICATIONS

Introduction

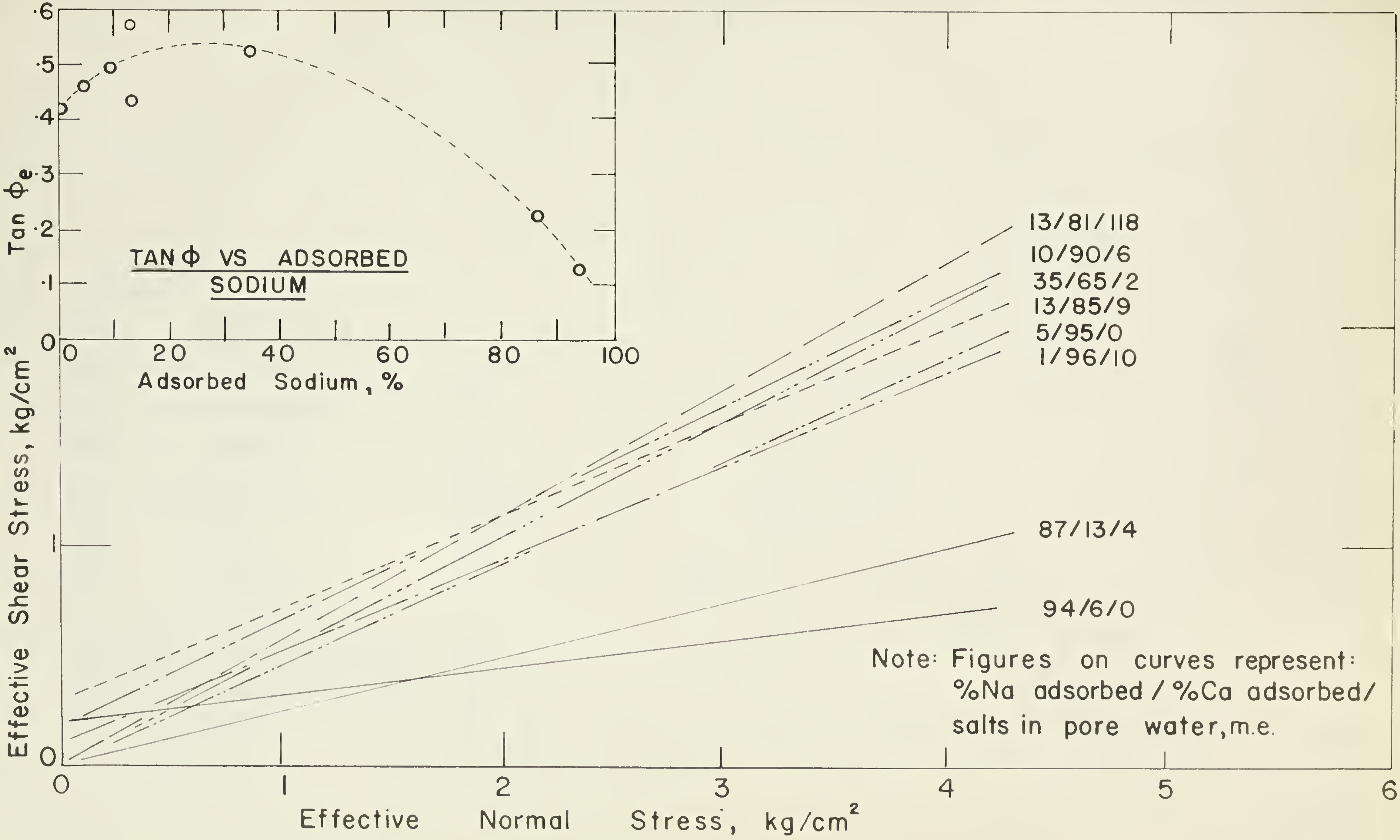
K.1 It has been reported (22) that the adsorbed water film on a homionic clay may not be affected by the adsorption of a second ion up to a certain point. As this certain critical point is passed there may be an abrupt change in the thickness of the adsorbed water film. This change is accompanied by corresponding strength changes. It was intended to investigate this phenomenon by having varying proportions of the adsorbed calcium cations of a homionic calcium clay replaced by sodium. Undrained triaxial tests with pore pressure measurements were used to determine the strength characteristics. The test series was stopped when the influence of the salt content of the pore water became apparent from the results of other tests. As a more rigourous investigation may be undertaken as a separate project, the author abandoned the investigation. However, the results are considered to be of sufficient interest to be reported. This appendix presents a brief outline of the procedures employed and the test results obtained.

Sample Preparation

K.2 The sodium-calcium modifications were prepared based on the data of Hamilton as presented in Figure D4 of Appendix D. As the first step, the clacium clay was prepared following the procedure outlined in Paragraph 4.2 of Chapter VII. The first batches were washed in alcohol in the centrifuge. After drying, the calcium soil was flooded with 500 ml of a sodium acetate solution. The normality of the sodium solution was chosen from Figure D4 after having decided on the amount of sodium to be adsorbed. In later batches, centrifuging was not carried out before adding the sodium acetate but instead three alcohol washes were added and settlement allowed to take place as in the preparation stages. This procedure was not successful due to the fact that a small amount of calcium remained in the pores which interfered with the exchange of sodium for calcium. Centrifuging to clear any extraneous ions from the pores is a necessary step and if carried out then the Langmuir equation will work. It must be kept in mind that the curve was set up for a 10;1 solution-soil ratio and for sodium replacing calcium. Any other combination would require the establishment of another curve.

Test Results

K.3 The results are summarized on Plates K1 and K2 of this appendix.



Note: Figures on curves represent: %Na adsorbed / %Ca adsorbed / salts in pore water, m.e.

PLATE K1 EFFECTIVE MOHR ENVELOPES FOR SODIUM-CALCIUM MODIFICATIONS

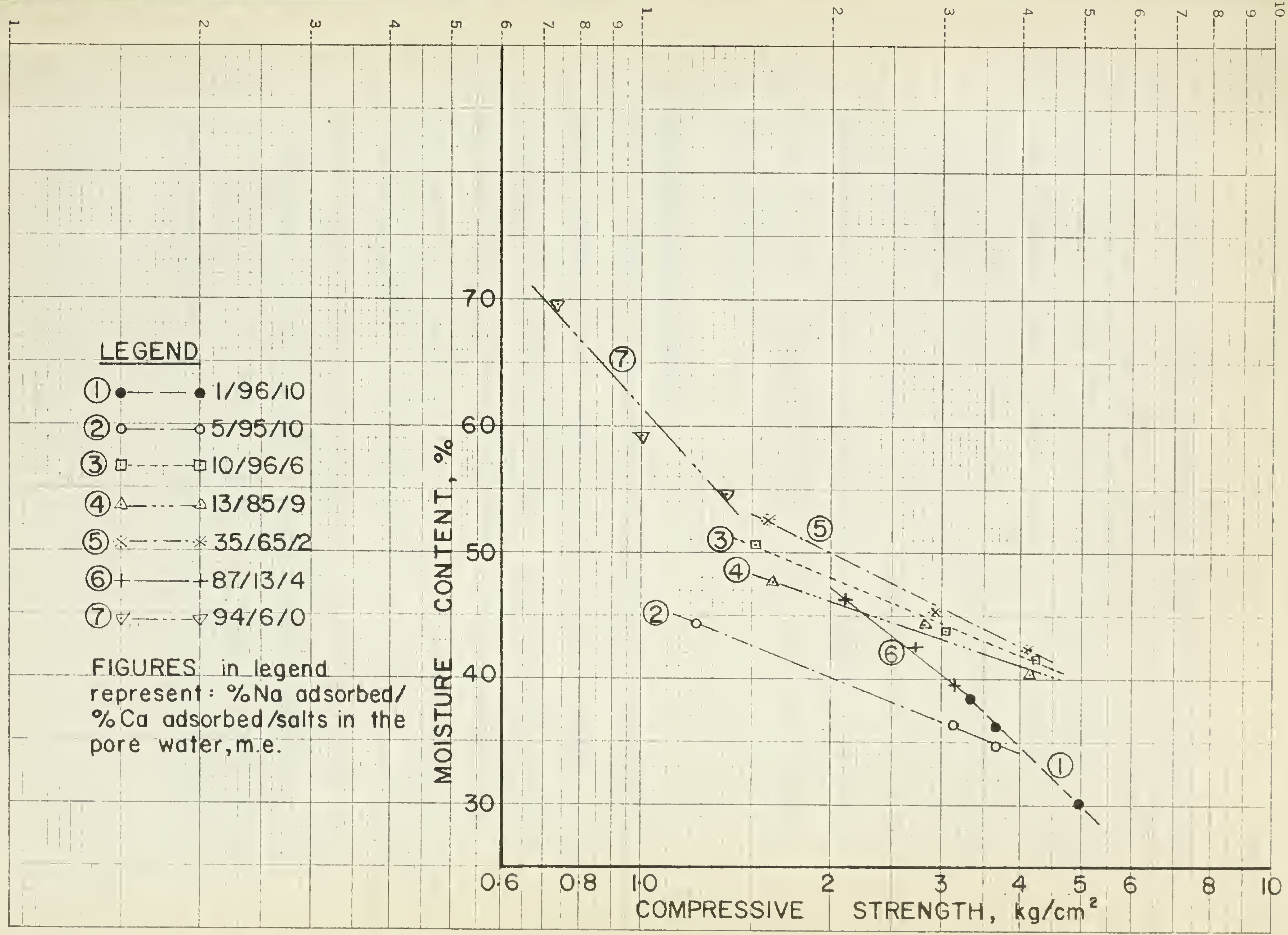


PLATE K2· MOISTURE CONTENT—STRENGTH RELATIONSHIPS FOR SODIUM-CALCIUM SOIL MODIFICATIONS

Flame photometer analysis of the adsorbed cations and the salt contents of the pore water of the samples are noted on the appropriate effective stress Mohr envelope of Plate K1. A group of three samples was used to establish each Mohr envelope. The four envelopes marked 94/6/0, 87/13/4, 1/96/10 and 13/81/118 were obtained from the main testing program and are drawn in for completeness. A graph of the tangent of the effective angle of internal friction versus the percent sodium adsorbed is drawn in the upper left hand corner. Plate K2 is a plot of the logarithm of the compressive strength versus the moisture content at the end of the test.

Discussion of Test Results

K.4 Although the results are inconclusive there is some indication that the maximum friction angle occurs when sodium occupies about 25 to 30 percent of the exchange positions. In the light of the postulates presented in the main body of this thesis, it follows that the thinnest adsorbed water films on the clay particles also occur at the same amount of adsorbed sodium. It would appear, therefore, that the replacement of adsorbed calcium by sodium leads to a gradual thinning of the adsorbed water film as the amount of sodium adsorbed increases. The actual mechanism causing the thinning of the water film around the clay particle is not known but one possible explanation may be as follows.

Due to electrical attractions and repulsions, the divalent calcium cation and the monovalent sodium cation enhance the oscillations of each other to the point where the structure of the adsorbed water film on the clay particle undergoes the maximum destruction. The optimum sodium-calcium interaction appears to occur when sodium occupies about 25 percent of the exchange positions. An increase of the amount of adsorbed sodium gradually brings about the characteristic thick water film associated with the adsorbed sodium cation.

K. 5 A study of Plate K2, log compressive strength versus moisture content, indicates that there is a tendency for the curves to shift upwards as the adsorbed sodium content increases from 5 to 35 percent. This is equivalent to saying that the moisture content increases for a given strength as the amount of adsorbed sodium increases from 5 to 35 percent. It is also of interest that all the curves on this graph are for the same soil. The only differences being brought about by changing the proportions of the ions occupying the exchange complex and varying the salt content of the pore water.

K.6 As has already been noted the results are inconclusive. The number of tests conducted are insufficient to establish reliably the relationship existing between the tangent of the effective angle of internal friction and the amount of sodium adsorbed. The variation of the salt

content from group to group is also complicating factor in this test series. For these two reasons the results of these tests must be left in this incomplete state pending further research.

APPENDIX L

RESULTS OF TESTS ON WASHED NATURAL SOIL

APPENDIX L

RESULTS OF TESTS ON WASHED NATURAL SOIL

Introduction

L.1 Subsequent to the completion of the main testing program of this thesis an opportunity became available to test the natural soil after removal of some of the salts in the pore water. The tests were carried out by G. Locker, a post-graduate student, under the direction of the author and were used as a familiarization series on the Geonor Triaxial Equipment for Mr. Locker as well as forming a part of his thesis. The series are a natural outcome of the general concept of strength variation with salt content and are appended since the data became available too late to be incorporated in the main body. Permission to use the data was kindly given by Mr. Locker.

Soil Preparation

L.2 The soil was prepared by placing 200 grams of minus No. 40 sieve material in each of 3 one quart kitchen sealers. Five hundred milliliters of distilled water were added and the soil brought into suspension with a milkshake machine. After settling, the clear supernatant

water was drawn off and replaced with fresh water. This was repeated a total of six times. Normal alcohol washing completed the treatment. The remainder of the testing followed the procedure outlined in Chapter IV.

Results of Tests

L.3 The Mohr plot and moisture content versus log compressive strength are contained in this appendix on Plates L1 and L2. The Mohr plot shows a decrease in the angle of internal friction to 19.5 degrees from 21 degrees as the salt content decreased from 73.8 me to 55.6 me. The cohesion changes very little when compared to the natural soil. The plot of moisture content versus log of compressive strength shows the line steepening as the salts are removed. The average time to theoretical 100 per cent consolidation for the washed natural soil is 280 minutes as compared to 150 minutes for the natural soil. (See Table 4).

Conclusions

L.4 The results of the tests on the soil modification achieved by removing a portion of the salts from the natural soil are in accord with the general conclusions reached during the thesis program and thus appear to substantiate the views presented therein.

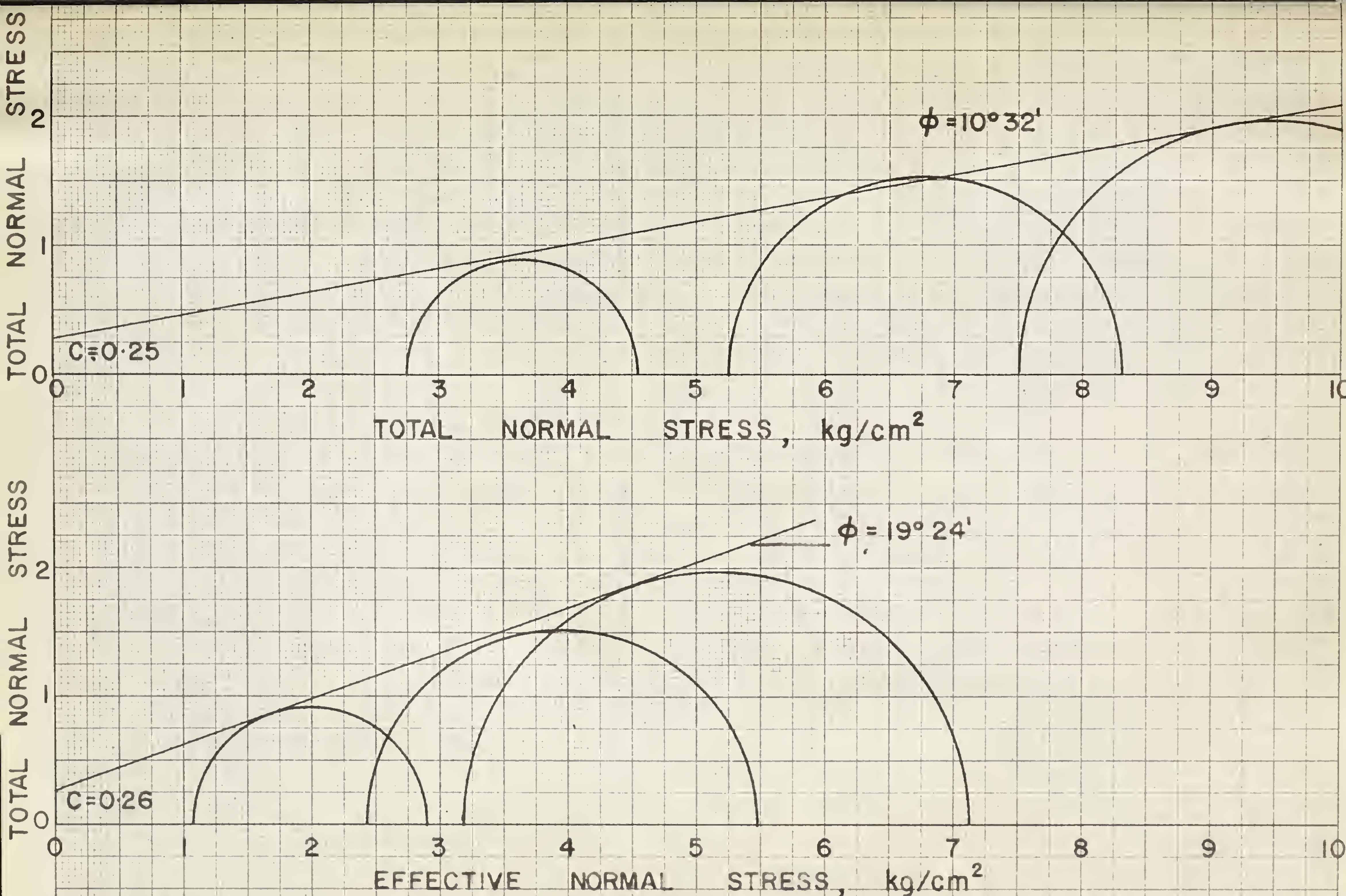
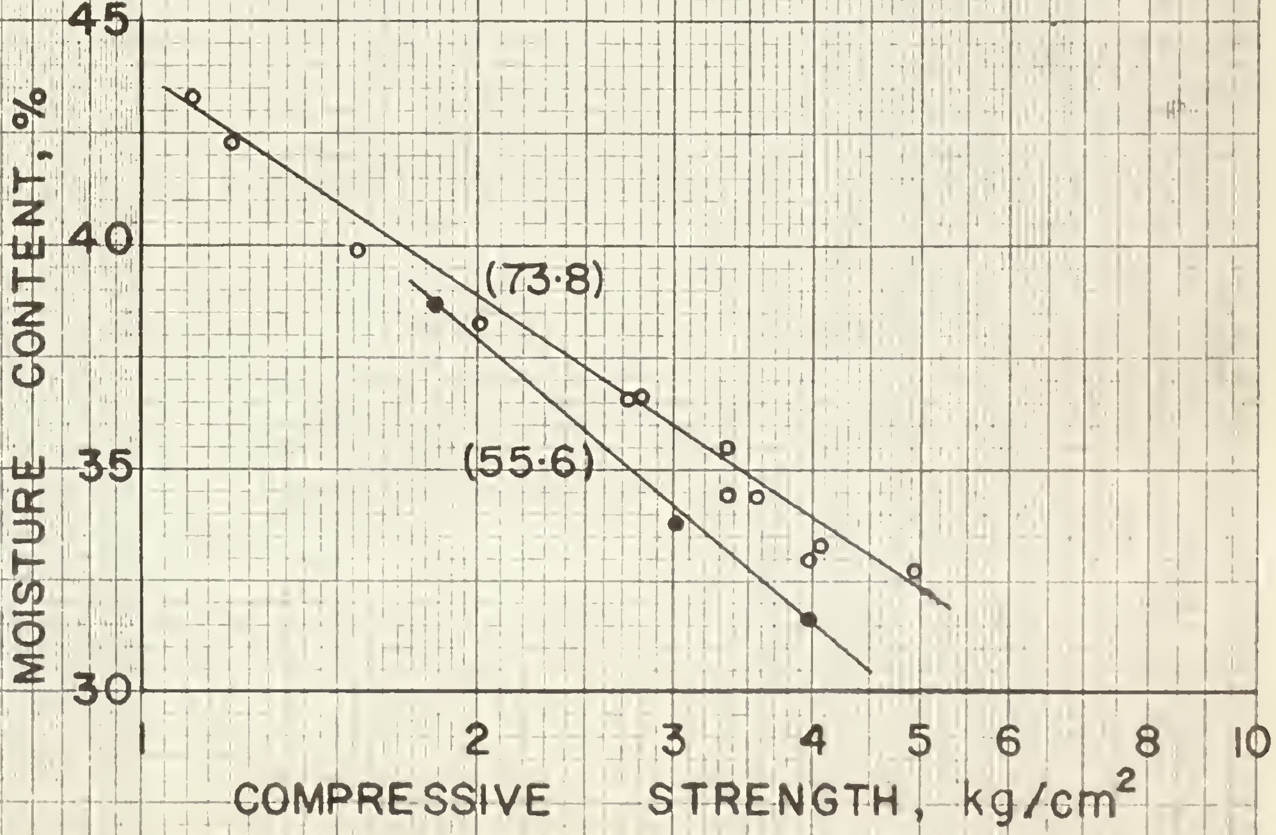


PLATE L1 MOHR ENVELOPES FOR NATURAL WASHED SOIL

PLATE L2

MOISTURE CONTENT - STRENGTH RELATIONSHIPS FOR NATURAL SOIL AND WASHED MODIFICATION



LEGEND:

- ○ Natural Soil
- ● Natural Soil after removal of a portion of the salts in the pore water

Salt contents of the pore water noted on curves. (m.e./100gm soil)

PLATE L2

B45463